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QUANTITIES OF MATERIALS AND COSTS PER SQUARE FOOT OF FLOOR FOR HIGHWAY AND ELECTRIC-RAILWAY LONG-SPAN SUSPENSION BRIDGES

By J. A. L. Waddell,* M. Am. Soc. C. E.

Synopsis

The ground covered by this paper is thoroughly and concisely explained by its title, which indicates even its limitations. Its object is to enable any engineer of good, general experience to compute, in two or three hours, when full preliminary data are furnished, the total cost of any projected highway or electric-railway or combined-highway-and-electric-railway suspension bridge and its approaches, including all the items of expense which, under ordinary conditions, would form the grand total cash outlay necessitated by the contemplated enterprise.

The paper is divided as follows into six distinct parts:

Introduction;
Assumed Data;
Findings;
Applications;
Deductions; and
Conclusion.

Under "Assumed Data" the following topics are listed and discussed more or less at length: (1) Roadway; (2) Live Loads; (3) Impact; (4) Wind Loads; (5) Floors; (6) Towers; (7) Cables; (8) Back-Stays; (9) Pier Shafts; (10) Pier Bases; (11) Anchorages; (12) Tower Encasement; (13) Lighting; (14) Materials; (15) Intensities of Working Stresses; (16) Unit Prices; and (17) Panel Lengths, Truss Depths, and Cable Sags.

Under the heading "Findings" are given by diagram, and discussed, the quantities of materials per square foot of floor in: (*a*) Flooring; (*b*) Floor Systems and Hand Rails; (*c*) Lateral Systems; (*d*) Stiffening Trusses; (*e*) Suspenders; (*f*) Main Cables; (*g*) Back-Stays; (*h*) Towers; (*i*) Pier Shafts; (*j*) Pier Bases; (*k*) Tower Encasement; (*l*) Lighting; (*m*) Anchorages; (*n*) Costs; and (*o*) Total Amounts of Metal.

The "Findings" show likewise by diagrams the quantities of all materials in steel trestle approaches to highway and electric-railway bridges of all kinds. Under this heading are treated also certain "Miscellaneous Costs" that cannot be included on the basis of "per square foot of floor," such as retaining walls, earth embankments, plazas, right of way, and property damages.

Note.—Written discussion on this paper will be closed in February, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Cons. Engr., New York, N. Y.

Under the heading "Applications," in order to demonstrate how the information furnished can be utilized, a specific case is worked out, all the necessary preliminary data being listed under the three divisions: (*p*) General; (*q*) Dimensions; and (*r*) Financial.

The computations made from these data are presented in the three succeeding groupings: (*s*) Cost Table for Main Span; (*t*) Cost of Approaches; and (*u*) Grand Total Cost of Project.

Under the heading "Deductions" there are discussed the suspending of the floor of side spans from the back-stays instead of using independent steel trestle approaches, the comparative costs of these two types of layout, and the determination of the extreme feasible lengths of suspension spans.

In the "Conclusion" the most important point raised is the writer's appeal to his brother engineers, not only to discuss this paper but also in their discussions to bring up and treat any other new matters of interest connected with suspension-bridge design and construction, in order to add materially to the general knowledge of the Engineering Profession concerning this important sub-division of the specialty of bridgework.

Introduction

In America, due to the wonderful development of the automobile, and to its use by all classes of people, especially in touring, highway bridges have become even more important than railway bridges. Moreover, on account of the exceedingly heavy motor trucks now used, both the live load and the dead load for the new structures are far greater than those in vogue a decade or two ago. The building of new post roads necessitates the bridging of the largest rivers of North America; and some of these are either so wide or so important for navigation as to require long spans.

As shown by comparative computations,* the limiting economic span length for highway cantilever bridges, pure and simple, as compared with the corresponding suspension bridges, was 1 000 ft. in 1918. Some engineers contend that it is shorter and others that it is longer; hence, probably in most cases this is not far from correct. However, the present falling price of steel wire for cables tends to lessen this span length of equal cost, as does also the adoption of the deflection method of computing moments and shears in the stiffening trusses, instead of the elastic method formerly used. For this reason that length has been chosen as the inferior limit for this investigation, the superior trial limit being the 5 000-ft. span. The curves, however, have been extended by extrapolation down to spans of 500 ft., and one of them up to a length of 7 000 ft.

Certain river crossings now contemplated demand spans well in excess of 1 000 ft., necessitating suspension bridges. Among others may be mentioned the one over San Francisco Harbor to connect San Francisco with Oakland and Berkeley, Calif., one to cross the Hudson River, connecting New York, N. Y., at 179th Street with a number of important municipalities in New

* "Economics of Bridgework," by J. A. L. Waddell.

Jersey; and one across The Narrows of New York Harbor to join Brooklyn, N. Y., and Staten Island.

As there are almost no reliable data concerning the costs of such long spans, the wildest kinds of guesses are constantly being made by laymen, and even by engineers. This condition has led the writer to make the present investigation.

During the last few years he has had occasion in his practice to compute in detail the costs of six proposed highway suspension bridges with span lengths of 1 550 ft., 1 750 ft., 1 975 ft., 2 500 ft., 2 800 ft., and 3 200 ft., in addition to numerous other computations.* Unfortunately, the widths of the decks and the live loads for these six bridges do not agree; consequently, it was necessary to make the computations methodically *de novo* and to use the results of all previous calculations merely for checking.

Assumed Data

Roadway.—For the sake of rigidity in both the span and the towers, the horizontal distance between the central planes of the trusses should be not less than one-thirty-fifth of the span. On this basis, adopting a clear roadway of 80 ft., with no sidewalks, for spans up to 3 300 ft., the width increases gradually thereafter to 129 ft., for a span of 5 000 ft.

No consideration has been given to the matter of sidewalks; for, without appreciable error, their cost per square foot may be computed as one-half of that for the main roadway. For instance, in the case of a bridge having an 80-ft. clear roadway and two sidewalks, each 10 ft. wide in the clear, the calculations should be made for an area of 90 sq. ft. per lin. ft. of span.

The spacing of the electric-railway tracks was taken as 10 ft., from center to center, or eight lines of track for an 80-ft. clear roadway carrying tracks only.

Live Loads.—For the deck and the suspenders the live loads adopted were taken from the writer's treatise, "Bridge Engineering", being Class A (with a 20-ton truck load, however) for the highway; Class C for the sidewalks; Class 30 for electric-railway stringers; and Class 25 for electric-railway cross-girders.

The live loads for stiffening trusses, cables, towers, and pier tops are shown in Fig. 1.

Impact.—The impact allowances used on floors and suspenders in both highway and electric-railway bridges are those given in Chapter VII of "Bridge Engineering" for highway structures. This assumption is valid, because recent investigations have proved that the impact in electric-railway bridges is far less than in the corresponding steam-railway structures, and, as nearly as may be, identical with that for the corresponding highway bridges. Owing to the great lengths of all trusses and cables, the impact upon them has been assumed to be zero.

Wind Loads.—The unit wind loads adopted were 30 lb. per sq. ft. for spans of 1 000 ft., reducing gradually to about 20 lb. per sq. ft. for those of 5 000 ft.

* "Economics of Bridgework," Chapter XIII.

The exposed area of vehicles was not considered for highway bridges, but for electric-railway bridges two lines of cars (one not in any way protecting the other) were assumed to be subject to wind pressure with 10 sq. ft. of exposed area per linear foot of track, reasonable lengths of trains being used.

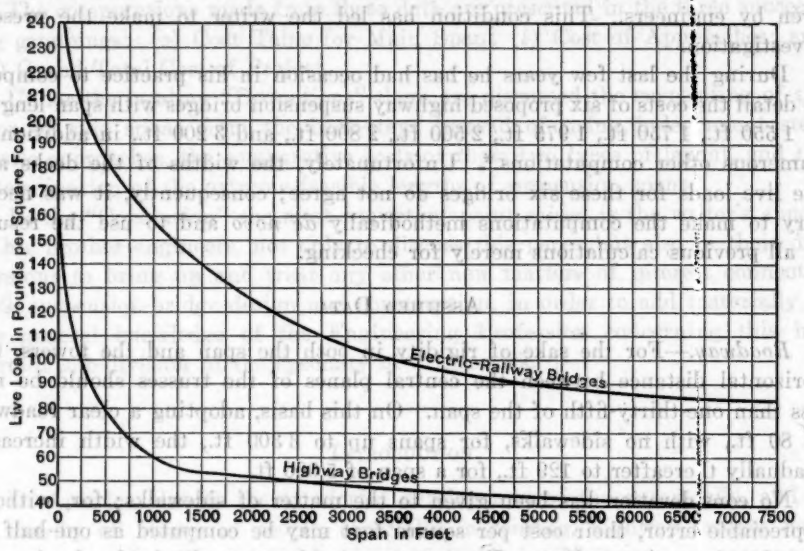

FIG. 1.—LIVE LOADS FOR SUSPENSION BRIDGES.

Fig. 2 records the wind pressures per linear foot of span for both highway and electric-railway suspension bridges. It will be noticed that for each type of bridge there are two lines of loading, the upper one being for the structure as a whole and the lower one for the trusses and laterals. Modern studies in suspension-bridge designing have shown that a part of the wind load is taken up by the cables and carried by them to the tops of the towers, and that the proportion thus carried increases with the span. That proportion is assumed to be a function of the quotient of the distance between central planes of trusses by the span. This assumption is probably only approximately correct;

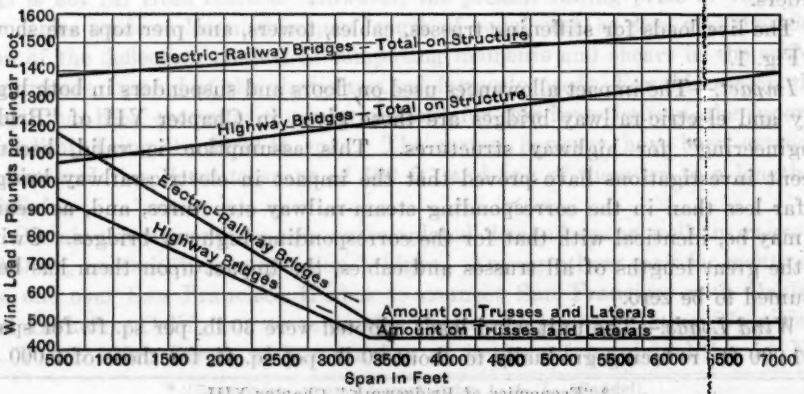

FIG. 2.—WIND LOADS FOR SUSPENSION BRIDGES.

but, in view of the present state of professional knowledge, it will have to suffice. It accounts for the horizontality of the lines in Fig. 2 beyond the span of 3 300 ft., where the width of structure begins to increase. Theoretically, there should be a slight upward tendency in these parts of the lines to correspond with the upward inclinations of the graphs for the total wind load on the structure; but such a refinement is not warranted, as it is claimed by certain authorities that in long spans the effect of wind pressure on trusses and laterals is hardly worthy of consideration, owing primarily to the massiveness of the cables and also to the rigidity of the wide concrete platform composing the highway deck. Be this as it may, it was not considered wise to cut down the effective wind loads on trusses and laterals any more than is indicated in Fig. 2, notwithstanding that it is never possible for great live loads and great wind loads to act simultaneously on any highway bridge, and seldom, if ever, on any electric-railway bridge.

Floors.—In long-span highway bridges it is economical to reduce the thicknesses and weights of the floor-slab and pavement to a minimum and thus the amount of metal throughout the structure (except in the stiffening trusses, which are not affected to any material extent by the dead load). This may be accomplished effectively in three ways, namely, by using metal liberally for reinforcement; by adopting a light aggregate for the concrete; and by making the pavement as thin as is consistent with due provision against its failure under heavy traffic.

In the Philadelphia-Camden Suspension Bridge, the use of a special system of reinforcement reduced the thickness of the base to 6 in.; and that is about the limit of feasibility, unless the spacing of the stringers be made ultra-close. In these computations a similar slab, 6½ in. thick, was adopted with 5-ft. stringer spacing.

During the World War, the Federal Government in building reinforced concrete ships used concrete weighing only from 110 to 120 lb. per cu. ft. There is also a light aggregate, known as "Haydite", made of clay or shale burned and aerated by a patented process that reduces the weight of plain concrete to 106 lb. per cu. ft. It is claimed that such concrete is stronger and more impervious to water than that composed of the ordinary aggregates; but these assertions need corroborative evidence. In making these computations, however, the writer has assumed that a satisfactory light aggregate will be available for building the floors of future suspension bridges.

In respect to bridge pavements, engineers are fast coming to the conclusion that a great thickness is not really necessary, but that a thin pavement of bituminous concrete can be used and replaced as it wears off. In these calculations a thickness of 1½ in. is assumed.

The electric-railway flooring adopted consists of the usual wooden ties and guard-rails and 90-lb. steel rails.

Towers.—The towers are flat at their bases and are rigidly attached to the masonry of the piers, the tops of which are about 25 ft. below clearance elevation to permit the feet of the columns to be spread transversely to the structure. Their dimensions are to be such as will permit their tops to spring

properly in the longitudinal direction of the bridge under all possible variations of temperature and loading. Theoretically, it would be more scientific to hinge them at the bottom. This would require less metal, but would involve some trouble and expense in staying them during erection.

Cables.—The cables are assumed to be of the best procurable quality of carbon-steel wire. Certain engineers are of the opinion that eye-bar cables are preferable. It is evident, however,* that no untreated eye-bars can compete with wire for cables, except perhaps in very short spans, and that heat-treated bars might be more economical than wire for fairly-long spans. A few years ago heat treatment for eye-bars was in an experimental stage, and wire cables were very expensive. Now heat treatment is claimed to have been well developed but, on the other hand, the cost of wire cables has been greatly reduced, and is still falling. Hence, until the prices of these materials have reached a condition of comparatively stable equilibrium, the economy of eye-bars over cables for any long-span suspension bridge is questionable.

Back-Stays.—For the purpose of this investigation the back-stays are assumed to be unloaded and to have the same inclination to the vertical as the main cable has at the tower top. Their lengths were made sufficient to bring their ends down to an elevation of 20 ft. above low water.

Pier Shafts.—The shafts of the piers are to be of plain concrete, extending down to an elevation 2 or 3 ft. below low water, where they will rest either on rock or on a base of timber and concrete.

Pier Bases.—When a shallow rock foundation is not available, the pier shafts are assumed to rest on pneumatic caissons sunk to bed-rock at a depth of about 100 ft. below low water—in other words, the bases are made 100 ft. high. This is likely to be their height in most cases, for the reason that, with bed-rock sloping down rapidly from the shore, the pier will probably be located, for the sake of economy, as far out as practicable; and the assumption made involves the maximum safe air pressure.

Anchorages.—Two types of anchorages have been provided, namely (*a*) those resting in the dry on bare bed-rock; and (*b*) those supported on fairly stiff clay, reinforced by long piles, and encased in a box 30 ft. deep filled with concrete.

For each of these types it is feasible to construct cost curves that will give comparatively accurate results. There is, however, still another case, that is, when the anchorage is obtained by tunneling into a hillside of solid rock; here the governing conditions will vary so greatly that it is not practicable to plot the costs of the anchorages. However, no great error would be made in assuming the same cost as for anchorages on bare rock. The structures have been computed on the assumptions of solid masses of concrete, of no attempt at decoration, and of cheap material to obtain weight inexpensively. If the anchorages are pierced by arched openings, if a stone masonry covering be used, or if other decorative features be adopted, the costs given by the diagrams will have to be materially increased.

* "Comparative Economics of Wire-Cables and High-Alloy-Steel Eye-Bar Cables for Long-Span Suspension-Bridge," *Proceedings*, Engrs. Soc. of Western Pennsylvania, Vol. 36, 1920–21, p. 418.

In designing the bases of the anchorages, no reliance was placed on the horizontal resistance of the piles, and the center of pressure was always kept within the middle third of the base. Each anchorage was made in two independent parts, one for each side of the structure. For anchorages on pile foundations, in plan each part is wide in front and narrow behind; and in elevation it is low in front and high behind.

Tower Encasement.—The tower encasement adopted consists of a shell of reinforced concrete surrounding, but never touching, the steel tower. It gives the appearance of a solid masonry shaft of a decidedly decorative character. Such a method of improving the esthetics of the bridge is legitimate, but, of course, by no means necessary.

Lighting.—An adequate system of electric lighting for each structure has been assumed—of a decorative nature to conform properly to the importance of the proposed bridge.

Materials.—For comparatively short stringers, rolled carbon-steel I-beams were assumed; but for the longer ones nickel-steel built-beams were adopted. That alloy was used also for the cross-girders and the stiffening trusses. In the lateral system the diagonals were generally computed as nickel steel; but carbon steel was used in the over-head bracing wherever mass, for rigidity rather than for mere strength, was the prime requisite.

In the suspenders and their connecting details nickel steel was used, and for the cables wire rope of the greatest strength attainable. For the towers silicon steel was adopted, because *per se* it is generally more economical than nickel steel, the great advantage of the latter being in the reduction of dead load—a consideration that is negligible in towers.

Intensities of Working Stresses.—The unit stresses used for proportioning steel members were as follows:

Carbon steel in tension	16 000 lb.
Silicon steel in tension	24 000 "
Nickel steel in tension	28 000 "
Wire cables in tension	70 000 "

For compression members the following formulas for allowable stress were used:

Carbon steel	$16\,000 - 60\,\frac{l}{r}$
Silicon steel	$24\,000 - 90\,\frac{l}{r}$
Nickel steel	$28\,000 - 105\,\frac{l}{r}$

For combinations of live load, dead load, and wind load, an increase of 30% in these intensities was permitted.

Unit Prices.—The schedule prices adopted for materials in place when computing the costs of suspension bridges per square foot of floor were as follows:

Carbon steel	$0.07 per lb.
Silicon steel in towers	0.09 " "

Nickel steel	$0.10	per lb.
Wire cables	0.18	" "
Floor-slab, including reinforcement	35.00	" cu. yd.
Pavement	3.00	" sq. yd.
Electric-railway track	6.00	" lin. ft.
Shafts of piers	20.00	" cu. yd.
Shafts of anchorages	15.00	" cu. yd.
Bases of pneumatic piers	35.00	" cu. yd.
Bases of pile anchorages	30.00	" cu. yd.
Piles below anchorage bases	1.25	" lin. ft.

Panel Lengths, Truss Depths, and Cable Sags.—Table 1 gives the assumed panel lengths, truss depths, and cable sags for the five span lengths on which the calculations of quantities of materials were made.

TABLE 1.—ASSUMED DIMENSIONS OF SUSPENSION BRIDGES.

Span length, in feet.	PANEL LENGTH, IN FEET.		TRUSS DEPTH, IN FEET.		SAG OF CABLE, IN FEET.	
	Highway.	Railway.	Highway.	Railway.	Highway.	Railway.
1 000	24.4	25	24	25	100	105
2 000	33.9	40	34	40	175	185
3 000	44.1	54.7	44	55	250	265
4 000	54.1	70.2	54	70	325	345
5 000	64.1	84.8	64	85	400	425

The fact that the panel length in every case has been made about equal to the truss depth is merely for convenience in computing, and is by no means a requisite; nor are the panel lengths themselves of any great importance, in so far as the weight of metal in the floor system is concerned, as will be shown.

Again, in case the panel lengths for the long spans for any reason appear to be too great, they can be divided by the adoption of sub-diagonals and intermediate verticals.

The quantities of materials and their costs are sub-divided as follows: (1) Flooring, including pavement; (2) metal in floor system and hand railings; (3) metal in lateral system; (4) metal in stiffening trusses; (5) metal in suspenders; (6) metal in main cables; (7) metal in back-stays; (8) metal in towers; (9) lighting; (10) pier shafts; (11) pier bases; (12) anchorages; and (13) tower encasement.

FINDINGS

Flooring.—For each square foot of highway deck there will be required 0.54 cu. ft. of reinforced concrete and 0.125 cu. ft. of pavement, weighing in combination 80 lb. The similar quantities for the electric-railway deck will be 5.5 ft. B. M. of timber and 6.6 lb. of rails and attachments, weighing in combination 30 lb.

Floor Systems and Hand-Rails.—To find the quantities of metal in the floor systems ten cases were estimated, for panels of different lengths in both

highway and electric-railway structures. Strange to say, up to spans of 3 300 ft.—the point at which the width of structure was increased—there was almost no variation from 46 lb. per sq. ft. of floor, regardless of either panel length or character of loading. The metal was nickel steel for spans of 2 000 ft. and more; one-half carbon and one-half nickel steel in spans of less length. Fig. 3 shows the results for both highway and electric-railway bridges.

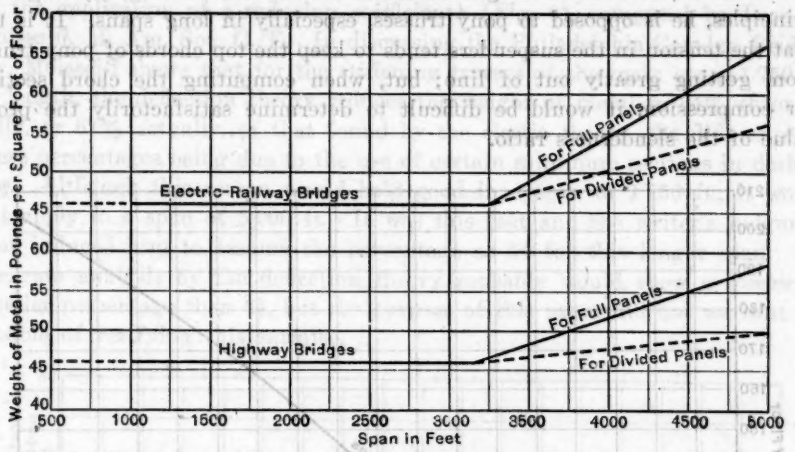

FIG. 3.—WEIGHTS OF METAL IN FLOOR SYSTEMS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

A valuable deduction may be drawn from these data as to the increase in weight of metal with an increase in the perpendicular distance between trusses.

If N is the percentage of increase in the width of the structure between trusses, X the corresponding percentage of increase in the weight of metal per square foot of floor for the floor system, and A is a constant, then:

$$X = A\,N \qquad (1)$$

For highway bridges the value of A is about 0.5 and for electric-railway bridges about 0.8.

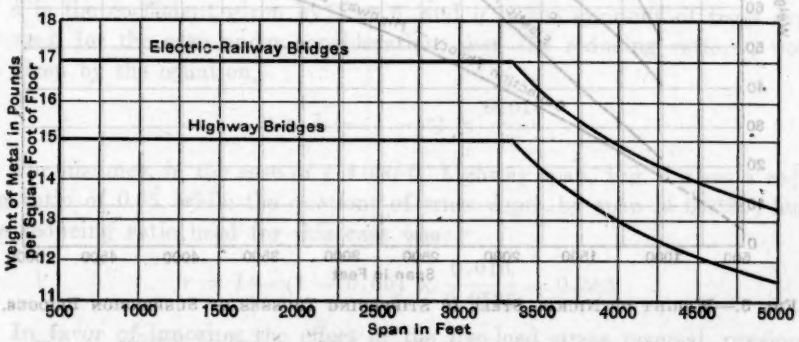

FIG. 4.—WEIGHT OF NICKEL STEEL IN LATERAL SYSTEMS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

Lateral Systems.—In Fig. 4 are recorded the weights of metal per square foot of floor found for the lateral systems of both highway and electric-railway

suspension bridges. In spans up to 2 000 or 2 500 ft., certain members of the lateral system may be built of carbon steel; but most of the weight recorded is that of nickel steel.

For short spans the weights may appear to be too high, especially when compared with those for the Philadelphia-Camden Bridge. The explanation is that in all the writer's designs he has used over-head bracing, as, on general principles, he is opposed to pony trusses, especially in long spans. It is true that the tension in the suspenders tends to keep the top chords of pony trusses from getting greatly out of line; but, when computing the chord sections for compression, it would be difficult to determine satisfactorily the proper value of the slenderness ratio.

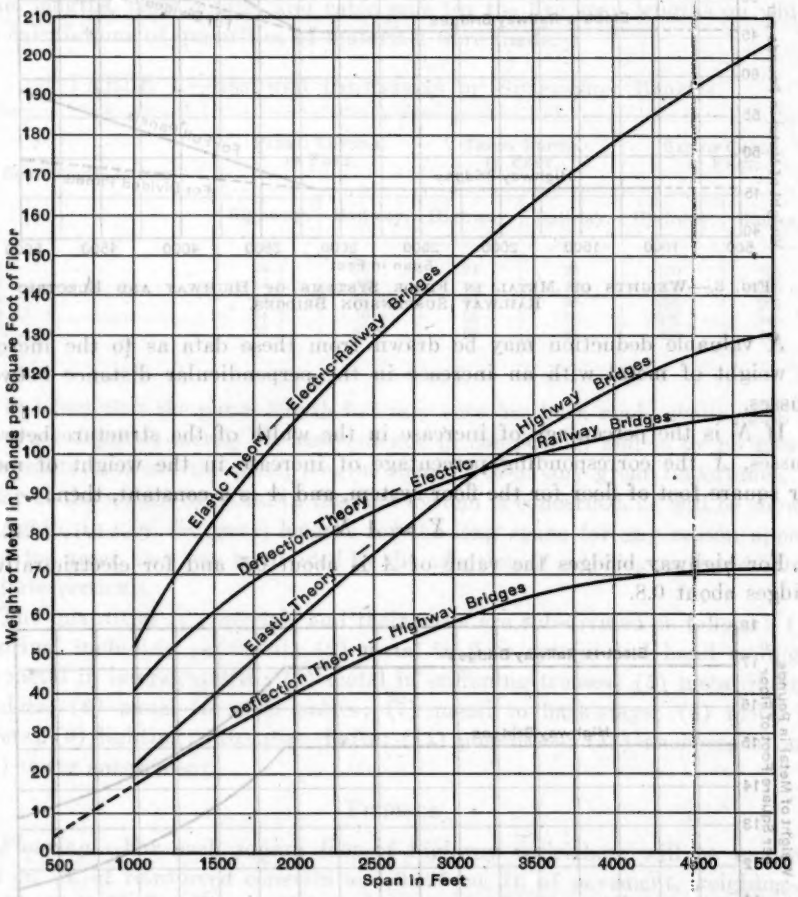
Weight of Metal in Pounds per Square Foot of Floor
Span in Feet
Elastic Theory — Electric-Railway Bridges
Deflection Theory — Electric-Railway Bridges
Elastic Theory — Highway Bridges
Deflection Theory — Highway Bridges

FIG. 5.—WEIGHT OF NICKEL STEEL IN STIFFENING TRUSSES OF SUSPENSION BRIDGES.

The peculiar peaks in Fig. 4 are due to the sudden widening of the structure, beginning at the span of 3 300 ft.; for, although the weight of metal per linear foot is increased slightly by the widening, the weight per square foot of floor is diminished.

Stiffening Trusses.—In Fig. 5 are indicated the weights of nickel steel for the stiffening trusses of both highway and electric-railway suspension bridges.

It will be noticed that there are two curves of weights for each of these types of structure. The upper one was computed by means of formulas based on the "elastic theory",* and the lower one was deduced from the upper one by the application of a reducing coefficient† (Fig. 6), computed by Leon S. Moisseiff, M. Am. Soc. C. E. In discussing the Philadelphia-Camden Bridge, Mr. Moisseiff shows that for the stiffening trusses of the main span, 1 750 ft. in length, the deflection theory reduces the weight of metal to 64% theoretically, or 67% actually, of that found by the elastic theory, the difference in these percentages being due to the use of certain minimum sections in designing. Although this reason would hold good in a span of 1 750 ft., it would not apply to a span of 5 000 ft. It was this fact and the writer's judgment that induced him to assume the percentage as 64 for this longer span. An accurate analysis by the deflection theory probably would show a somewhat smaller percentage than 64, but the purpose of this paper did not warrant the trouble of verifying this surmise.

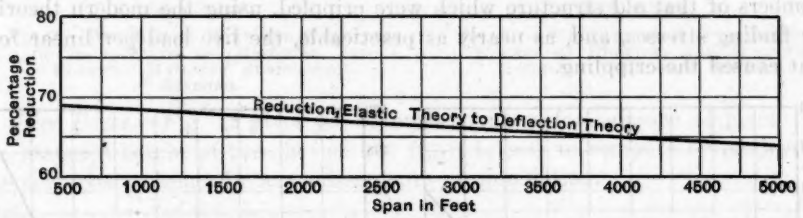

FIG. 6.—PERCENTAGES OF REDUCTION IN WEIGHTS OF METAL IN STIFFENING TRUSSES OF SUSPENSION BRIDGES AS FOUND BY THE ELASTIC THEORY TO AGREE WITH THOSE FOUND BY THE DEFLECTION THEORY.

Another assumption is that although the ratios for weight reduction found by Mr. Moisseiff in the case of the Philadelphia-Camden Bridge, in which the quotient of truss depth by span is 0.016, are correct for all span lengths where that quotient prevails, they would not hold good for other proportions. If c is the coefficient given by Fig. 6, and q is the quotient of truss depth by span for the case under consideration, then the reducing ratio, r, would be given by the equation,

$$r = 1 - (1 - c) \times \frac{0.016}{q}$$

For instance, in the case of a 4 000-ft. highway span, Fig. 6 gives a reducing ratio of 0.65, while the quotient of truss depth by span is 0.0135; hence the reducing ratio used for this case was:

$$r = 1 - (1 - 0.65) \times \frac{0.016}{0.0135} = 0.585$$

In favor of ignoring the effect of the live-load stress reversal, previously alluded to, there is much to be said:

* "Economics of Bridgework," Chapter XXIX.

† "The Towers, Cables, and Stiffening Trusses of the Bridge over the Delaware River, between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

First.—The latest long-span suspension bridges have been thus designed, consequently the pace has been set for bridge designers to follow, unless they feel that a mistake has been made in so doing.

Second.—When the greatest live load is on a highway bridge, it is generally distributed almost uniformly, in which case no reversion will exist; in an electric-railway structure the stress reversal is much more probable.

Third.—When stress reversal does occur, there is sure to be a considerable interval of time between reversals, even under electric-railway loading, hence the metal will have an opportunity to recover before being stressed in the opposite direction.

On the other hand, modern designing of stiffening trusses has a single trend, namely, toward cutting down sectional areas; and such economizing may be carried too far. It must not be forgotten that some of the truss members of the Brooklyn Bridge have crumpled under live load without the simultaneous existence of any wind pressure worth mentioning; hence, before an engineer starts the actual designing of any long-span suspension bridge, he should make some computations to determine the actual unit stresses in those members of that old structure which were crippled, using the modern theories for finding stresses, and, as nearly as practicable, the live load per linear foot that caused the crippling.

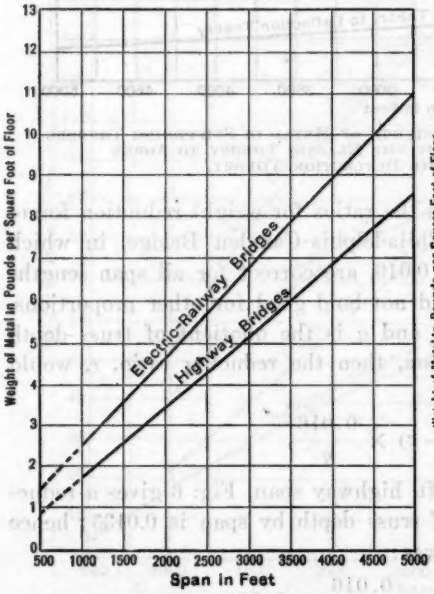

FIG. 7.—WEIGHTS OF NICKEL STEEL IN SUSPENDERS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

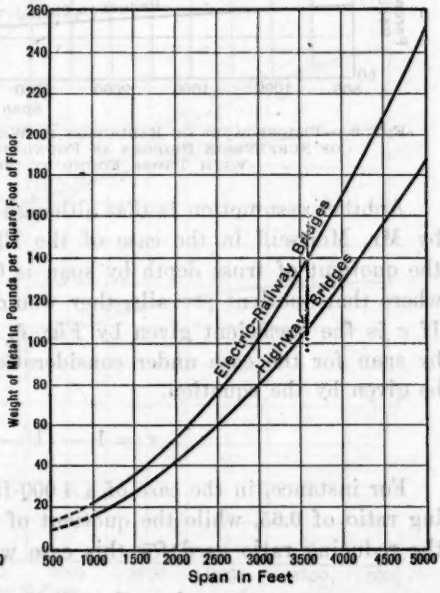

FIG. 8.—WEIGHTS OF WIRE AND WRAPPING IN MAIN CABLES OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

Wire Cables and Towers.—In Figs. 7 to 11 are given the quantities in suspenders, cables, back-stays, towers, and pier-shafts. Fig. 9, referring to the back-stays, is based on the assumption that they carry no vertical load

other than their own weight. When the back-stays do carry vertical loads by means of suspenders, this diagram will not be needed, as Fig. 8 only should then be used.

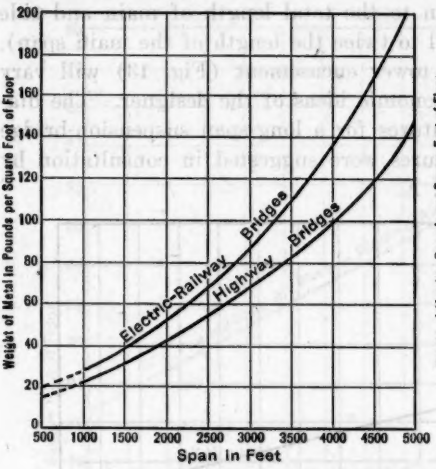

FIG. 9.—WEIGHTS OF WIRE AND WRAPPING IN BACK-STAYS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

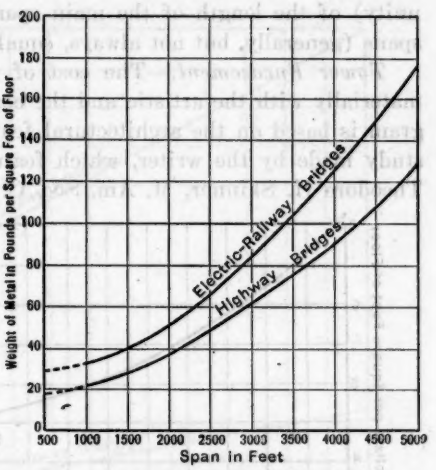

FIG. 10.—WEIGHTS OF SILICON STEEL IN TOWERS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

Pier Bases.—Fig. 12 gives the volumes of bases for caissons of piers. As the assumed height of base is just 100 ft., it is easy to compute by proportion the corrected volume for any other depth of foundation.

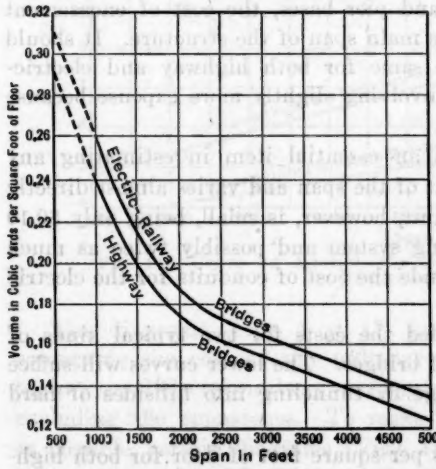

FIG. 11.—CONCRETE IN SHAFTS OF PIERS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

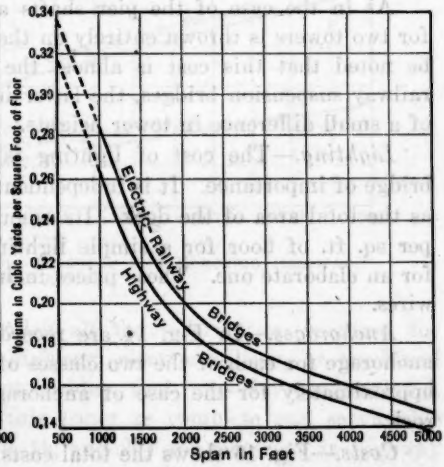

FIG. 12.—BASES OF PNEUMATIC OR OPEN DREDGING PIERS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

Figs. 11 and 12 were plotted on the assumption that there were no side spans hung from the back-stays, thus throwing the cost of the two piers in with that of the main span. If, however, the side spans are supported by the

back-stays, the total cost of the main piers will not be increased, as the loadings on the pier tops are practically the same for the two cases; but the quantities given on the diagrams will have to be multiplied by the ratio (less than unity) of the length of the main span to the total length of main and side spans (generally, but not always, equal to twice the length of the main span).

Tower Encasement.—The cost of tower encasement (Fig. 13) will vary materially with the artistic and the economic ideas of the designer. The diagram is based on the architectural features for a long-span suspension-bridge study made by the writer, which features were suggested in consultation by Theodore H. Skinner, M. Am. Soc. C. E.

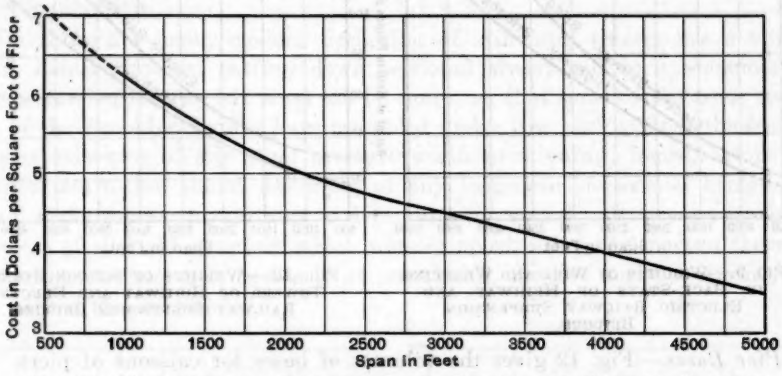

FIG. 13.—COST OF TOWER ENCASEMENT FOR HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

As in the case of the pier shafts and pier bases, the cost of encasement for two towers is thrown entirely on the main span of the structure. It should be noted that this cost is almost the same for both highway and electric-railway suspension bridges, the latter involving slightly more expense because of a small difference in tower heights.

Lighting.—The cost of lighting is an essential item in estimating any bridge of importance. It is independent of the span and varies almost directly as the total area of the deck. Its amount, however, is small, being only $0.10 per sq. ft. of floor for a simple lighting system and possibly twice as much for an elaborate one. These prices include the cost of conduits for the electric wires.

Anchorages.—In Fig. 14 are recorded the costs for two typical kinds of anchorage for each of the two classes of bridges. The lower curves will suffice approximately for the case of anchorage by tunneling into hillsides of hard rock.

Costs.—Fig. 15 shows the total costs per square foot of floor for both highway and electric-railway suspension bridges, each type having anchorages resting on dry rock and on piles. As the tower encasement is just as likely to be omitted as adopted, its cost has not been included in this diagram.

Total Amounts of Metal.—To answer the question "How much steel is required" for any particular suspension bridge, Figs. 16 and 17 have been prepared. Fig. 16 gives the weights of structural steel and of wire-cable

steel for suspension bridges, in which the side spans are not hung from the back-stays. Fig. 17 gives the corresponding weights when the back-stays carry the side spans.

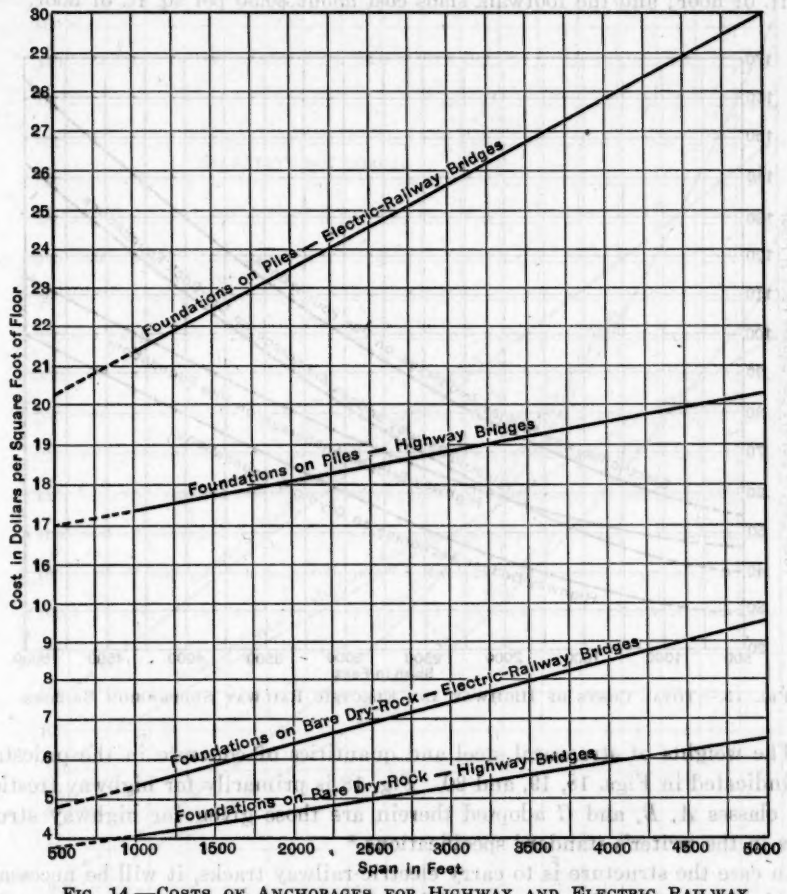

FIG. 14.—COSTS OF ANCHORAGES FOR HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

The diagrams, Figs. 1 to 17, furnish all the information required for computing the cost of any ordinary highway, electric-railway, or combined highway-and-electric-railway suspension bridge that may be contemplated, excluding the approaches. To make this paper as complete and serviceable as possible, certain diagrams (Figs. 18, 19, and 20) for materials in highway bridge approaches taken from the writer's office records, hitherto unpublished, will be included.

Steel trestle construction, here assumed, is the cheapest kind of approach to a bridge; hence, if reinforced concrete, or arches, or any unusual or fanciful kind of construction be used, the approaches will cost considerably more than these three diagrams indicate.

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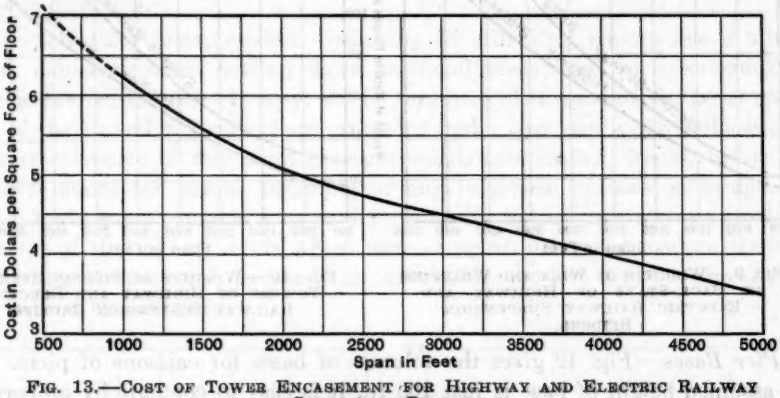

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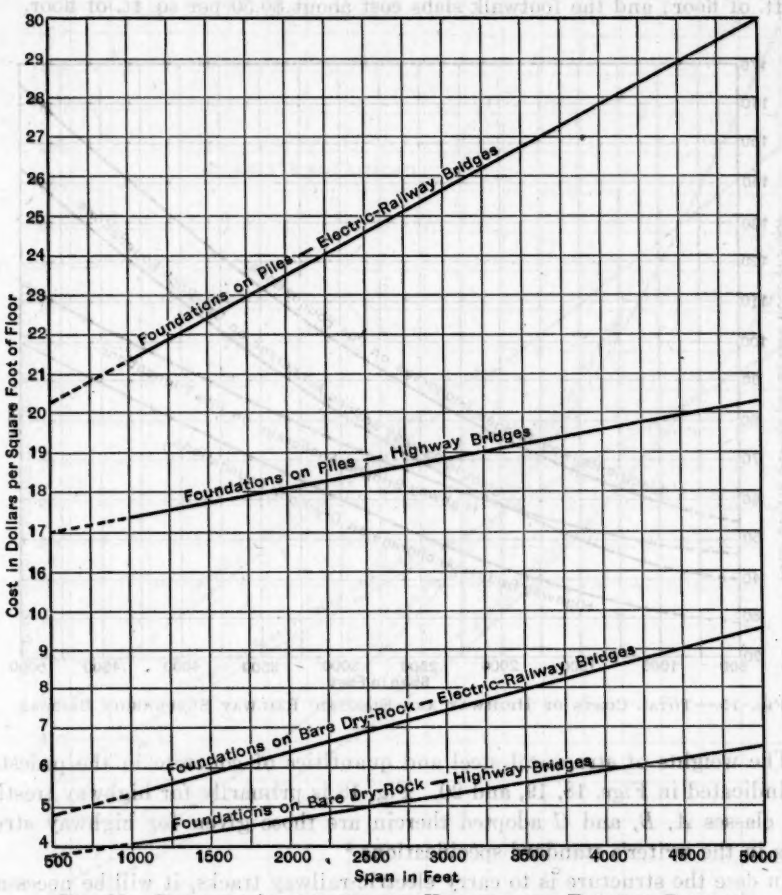

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Steel trestle construction, here assumed, is the cheapest kind of approach to a bridge; hence, if reinforced concrete, or arches, or any unusual or fanciful kind of construction be used, the approaches will cost considerably more than these three diagrams indicate.

In a highway steel trestle the roadway pavement, with its supporting reinforced concrete base, costs, including the reinforcement, about \$1.00 per sq. ft. of floor; the open deck of an electric-railway trestle costs about \$0.70 per sq. ft. of floor; and the footwalk slabs cost about \$0.50 per sq. ft. of floor.

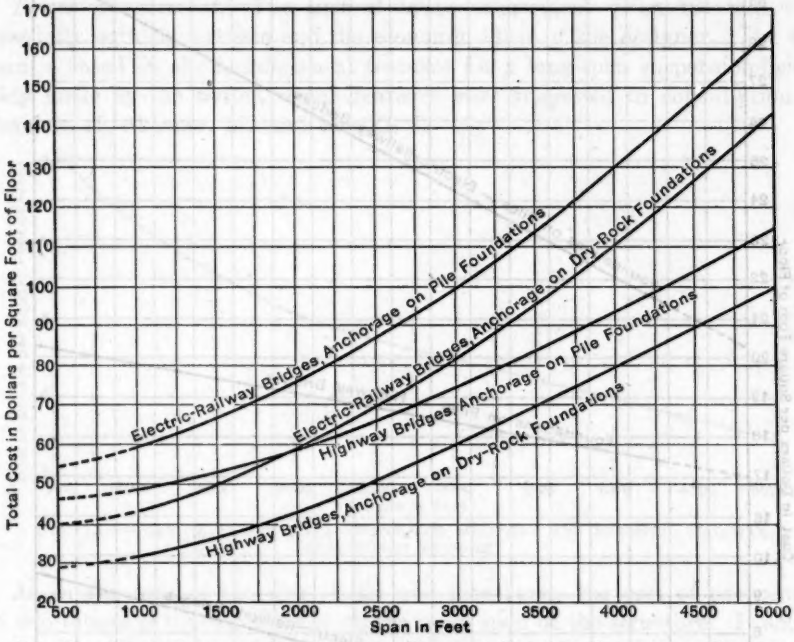

FIG. 15.—TOTAL COSTS OF HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

The weights of structural steel and quantities of concrete in the pedestals are indicated in Figs. 18, 19, and 20. Fig. 18 is primarily for highway trestles. The classes *A*, *B*, and *C* adopted therein are those given for highway structures in the writer's standard specifications.*

In case the structure is to carry electric-railway tracks, it will be necessary to add $(11 + 0.12h)$ lb. per sq. ft. of floor for the portion covered by the tracks, allowing a 10-ft. width for each track. If cantilevered footwalks are added, the weight of metal, w, per square foot for the additional area can be found by the formula, $w = 20 + 0.15h$, in which, h is the height of the trestle, in feet.

In case foundation piles are required for the pedestals, the number needed will be one for about every 80 sq. ft. of deck, irrespective of the length of the pile. This estimate is based on the use of small piles, each carrying 15 tons. These piles should generally be of reinforced concrete (about 12 in. square, or sometimes more). If wooden piles are used, they must be thoroughly protected from the atmosphere, and it will be necessary to increase somewhat the heights of the pedestals. For any change in the heights shown on Figs. 18 and 19 the quantities of concrete will vary almost proportionately.

* See "Bridge Engineering."

The lighting of the approaches will cost, as in the case of the bridge proper, about \$0.10 or \$0.15 per sq. ft. of floor.

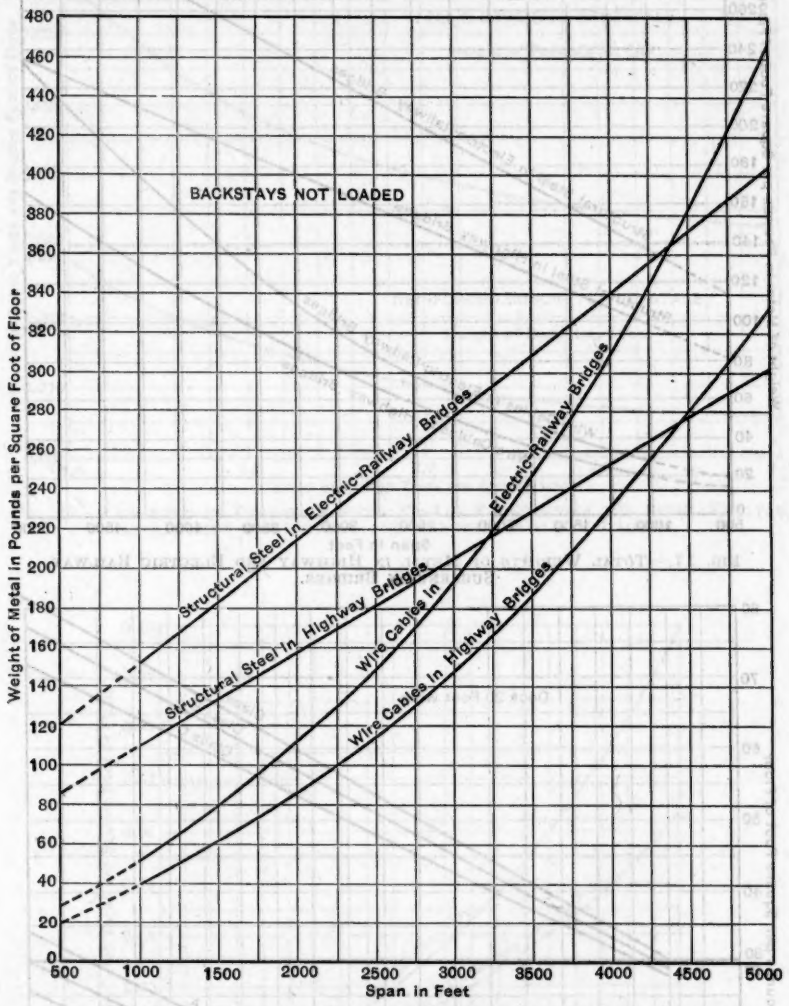

FIG. 16.—TOTAL WEIGHT OF METAL IN HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

Miscellaneous Costs.—Some items of cost for a complete estimate cannot be included on a "per square foot of floor" basis, such as the retaining walls at the low ends of the trestle approaches, the earth embankments with their paving, the plazas for the fare-collecting booths of toll structures, right of way, and property damages. These items will have to be allowed for, at least approximately, in any estimate.

The cost of a retaining wall usually will lie between \$100 and \$125 per lin. ft.; and its equalized length can be taken as the full width of roadway and sidewalks plus 50 ft.

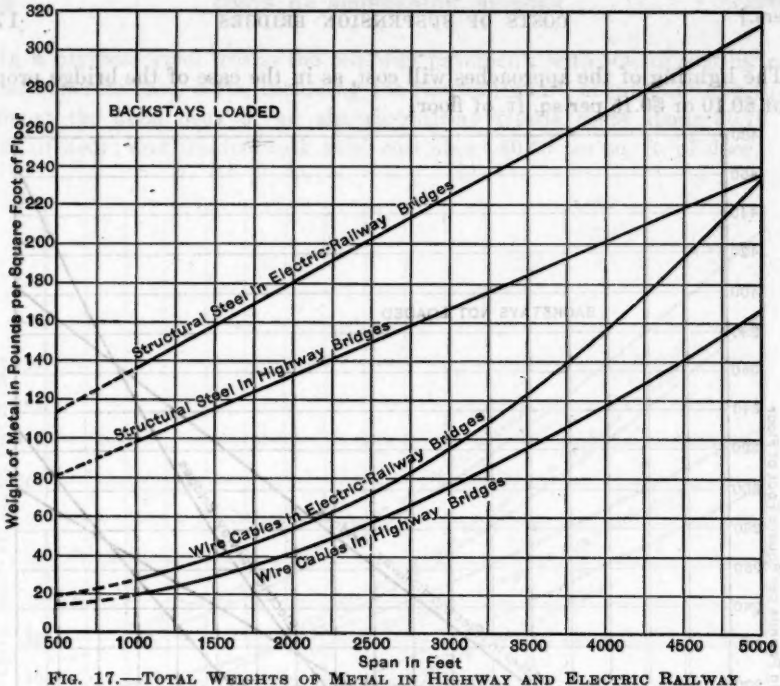

FIG. 17.—TOTAL WEIGHTS OF METAL IN HIGHWAY AND ELECTRIC RAILWAY SUSPENSION BRIDGES.

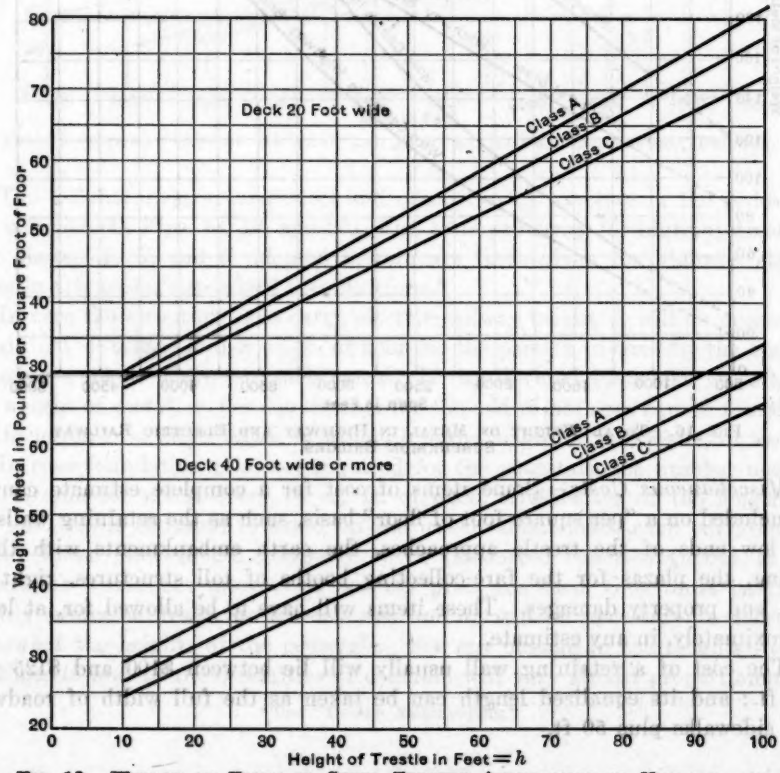

FIG. 18.—WEIGHT OF FLOOR IN STEEL TRESTLE APPROACHES TO HIGHWAY AND ELECTRIC RAILWAY BRIDGES.

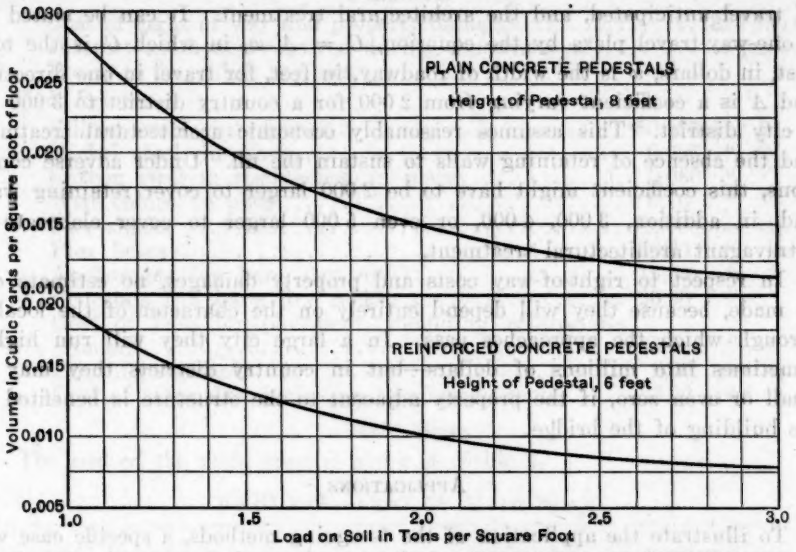

FIG. 19.—CONCRETE IN PEDESTALS WITH SPREAD FOUNDATIONS FOR STEEL TRESTLE APPROACHES TO HIGHWAY AND ELECTRIC RAILWAY BRIDGES.

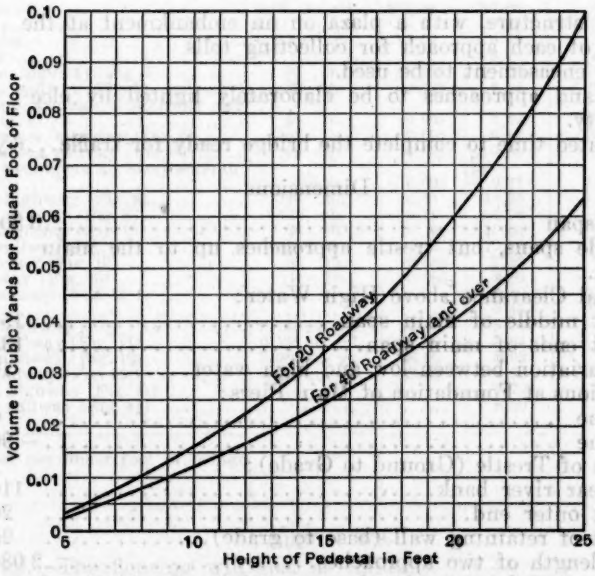

FIG. 20.—VOLUMES OF CONCRETE IN PEDESTALS ON ROCK FOUNDATIONS FOR STEEL TRESTLE APPROACHES TO HIGHWAY AND ELECTRIC RAILWAY BRIDGES.

The cost of the plazas will depend greatly on the location, the amount of travel anticipated, and the architectural treatment. It can be stated for a one-way-travel plaza by the equation, $C = A\ w$, in which C is the total cost, in dollars, w is the width of roadway, in feet, for travel in one direction, and A is a coefficient varying from 2 000 for a country district to 3 000 for a city district. This assumes reasonably economic architectural treatment and the absence of retaining walls to sustain the fill. Under adverse conditions, this coefficient might have to be 2 000 larger to cover retaining walls and, in addition, 3 000, 4 000, or even 5 000 larger to cover elaborate or extravagant architectural treatment.

In respect to right-of-way costs and property damages, no estimate can be made, because they will depend entirely on the character of the locality through which the approaches pass. In a large city they will run high—sometimes into millions of dollars—but in country districts they may be small or even zero, if the property adjacent to the structure is benefited by the building of the bridge.

Applications

To illustrate the application of the foregoing methods, a specific case will be assumed and computations made for the completed structure, according to the following dimensions and ruling conditions:

General

A toll structure, with a plaza on an embankment at the end of each approach for collecting tolls.
Tower encasement to be used.
Span and approaches to be elaborately lighted by electricity.
Estimated time to complete the bridge ready for traffic...6 years

Dimensions

Main span	3 800 ft.
No side spans, but trestle approaches up to the main span.	
Vertical Clearance above High Water:	
At middle of main span	150 ft.
At ends of main span	135 ft.
Variation between low and high water	10 ft.
Elevations at Foundation of Main Piers:	
One	0 ft.
One	—90 ft.
Height of Trestle (Ground to Grade):	
Near river bank	110 ft.
At outer end	20 ft.
Height of retaining wall (base to grade)	25 ft.
Total length of two approaches	2 080 ft.
Width of roadway in the clear, containing four lines of electric-railway tracks with open floor, located at mid-width	100 ft.
Width of sidewalk in the clear, each side cantilevered beyond the trusses	10 ft.

Financial

Cost of right of way and property damages.....$ 3 550 000
Interest on money required during construction..... 5 per cent.
Bonds to be sold at par, net.

Unit Prices.—

Silicon steel, in place	$0.085	per lb.
Nickel steel, in place	0.095	" "
Carbon steel in approaches, in place	0.060	" "
Wire cables, in place	0.16	" "
Pier shafts	22.00	per cu. yd.
Pier bases	38.00	" " "
Anchorages (on bare, dry rock)	15.00	" " "
Pedestals (on bare, dry rock)	20.00	" " "
Cost of base for pavement, including reinforcing metal	33.00	" " "
Cost of pavement	2.00	per sq. yd.
Cost of sidewalks, including reinforcing metal	33.00	per cu. yd.

Main Span

The cost of the main span is given in Table 2.

TABLE 2.—COST OF MAIN SPAN.

Item.	Quantity per square foot of roadway.	Unit price.	Product.	Equivalent area, in square feet per linear foot.	Cost per linear foot of span.
Highway slab	0.02 cu. yd.	$33.00	$0.66	60	$39.60
Pavement	0.11 sq. yd.	2.00	0.22	60	13.20
Sidewalk slabs	0.012 cu. yd.	33.00	0.40	20	8.00
Railway floor	.....	.....	0.70	40	28.00
Floor system, highway (Fig. 3)	50 lb.	0.095	4.75	70	332.50
Floor system, railway (Fig. 3)	52 lb.	0.095	4.94	40	197.60
Lateral system, average of highway and railway (Fig. 4)	16 lb.	0.095	1.52	110	167.20
Stiffening trusses, highway (Fig. 5)	67 lb.	0.095	6.37	70	445.90
Stiffening trusses, railway (Fig. 5)	102 lb.	0.095	9.69	40	387.60
Suspenders, average of highway and railway (Fig. 7)	7.5 lb.	0.095	0.71	110	78.10
Main cables, highway (Fig. 8)	116 lb.	0.16	18.56	70	1 299.20
Main cables, railway (Fig. 8)	156 lb.	0.16	24.96	40	998.40
Back-stays, highway (Fig. 9)	94 lb.	0.16	15.04	70	1 052.80
Back-stays, railway (Fig. 9)	124 lb.	0.16	19.84	40	793.60
Towers, highway (Fig. 10)	85 lb.	0.085	7.22	70	505.40
Towers, railway (Fig. 10)	114 lb.	0.085	9.69	40	387.60
Tower encasement (Fig. 13)	.....	.....	4.10	110	451.00
Lighting	.....	.....	0.15	120	18.00
Pier shafts, highway (Fig. 11)	0.141 cu. yd.	22.00	3.10	70	217.00
Pier shafts, railway (Fig. 11)	0.152 cu. yd.	22.00	3.34	40	133.60
Pier bases, highway (Fig. 12)	0.069 cu. yd.	38.00	2.62	70	183.40
Pier bases, railway (Fig. 12)	0.073 cu. yd.	38.00	2.77	40	110.80
Anchorages, highway (Fig. 14)	.....	.....	5.70	70	399.00
Anchorages railway (Fig. 14)	.....	.....	8.20	40	328.00
Total cost per linear foot of bridge proper	.....	.....	.....	.....	$8 375.50

Approaches

Flooring.—The flooring will cost as follows:

Roadway, 60 sq. ft., @ $1.00	$60.00
Sidewalks, 20 sq. ft., @ $0.50	10.00
Railway, 40 sq. ft., @ $0.70	28.00
Total cost of flooring per linear foot of structure	$98.00

Lighting.—

Cost per linear foot of structure..... $18.00

Trestle 110 Ft. High.—

Unit Quantities:

Metal (Class A), (Fig. 18), for roadway.....	80 lb.
Metal (Classes 25 and 30), (Fig. 18), for railway....	104 lb.
Metal (Class C), (Fig. 18), for sidewalks.....	36 lb.
Metal hand-rails, 4 @ 70 lb. per lin. ft.....	280 lb.

Pedestals, 9 ft. high on rock (Fig. 20), 0.01 cu. yd. per sq. ft. of floor.

Quantities per Linear Foot:

Roadway, 60 sq. ft. @ 80 lb.....	=	4 800 lb. per lin. ft.
Railway, 40 sq. ft. @ 104 lb.....	=	4 160 " " " "
Sidewalks, 20 sq. ft. @ 36 lb.....	=	720 " " " "
Hand-rails, 4 lines @ 70 lb.....	=	280 " " " "
Total weight of metal.....		9 960 " " " "
Say		10 000 " " " "

Pedestals, $0.01 \times 120 = 1.2$ cu. yd. per lin. ft.

Summary of Cost per Linear Foot of Trestle:

Metal, 10 000 lb. @ $0.06.....	$600.00
Flooring	98.00
Lighting	18.00
Pedestals, 1.2 cu. yd. @ $20.00.....	24.00
Total cost per linear foot of trestle 110 ft. high.....	$740.00

Trestle 20 Ft. High.—

Unit Quantities:

Metal (Class A), (Fig. 18), for roadway....	33 lb.
Metal (Classes 25 and 30), (Fig. 18), for railway.....	47 lb.
Metal (Class C), (Fig. 18), for sidewalks..	23 lb.
Metal hand-rails, 4 @ 70 lb.....	280 lb. per lin. ft.

Pedestals, 5 ft. high, on rock (Fig. 20), 0.002 cu. yd. per sq. ft. of floor.

Quantities per Linear Foot:

Roadway, 60 sq. ft. @ 33 lb.....	=	1 980 lb. per lin. ft.
Railway, 40 sq. ft. @ 47 lb.....	=	1 880 " " " "
Sidewalks, 20 sq. ft. @ 23 lb.....	=	460 " " " "
Hand-rails, 4 lines @ 70.....	=	280 " " " "
Total weight of metal.....		4 600 " " " "

Pedestals, $0.002 \times 120 = 0.24$ cu. yd. per lin. ft.

Summary of Cost per Linear Foot of Trestle:

Metal, 4 600 lb. @ $0.06.....	$276.00
Flooring	98.00
Lighting	18.00
Pedestals, 0.24 cu. yd. @ $20.....	4.80
Total cost per linear foot of trestle 20 ft. high.....	$396.80
Say	$400.00

Average Cost of Trestle.—

Cost per linear foot for 110 ft. height	$740.00
" " " " " 20 ft. "	400.00
	$1 140.00
Average cost per linear foot for trestle	$ 570.00

Retaining Walls.—

Length, 2 × (120 + 50)	340 ft.
Cost per linear foot	$ 100
Total cost of retaining walls	$34 000

Grand Total Cost of Project

Main structure, 3 800 ft. @ $8 375	$31 825 000
Approaches, 2 080 ft. @ $570	1 186 000
Retaining walls	34 000
Two plazas, 30 × 3 000	180 000
Right of way and property damages	3 550 000
Summation	$36 775 000
Contingencies at 5%	1 838 000
Summation	$38 613 000
Engineering and administration at 8%	3 089 000
Summation	$41 702 000
Interest during construction at 15%	6 255 000
Grand total cost of project	$47 957 000
Say	$48 000 000

Deductions

In case there are side spans with decks and stiffening trusses suspended from the cables, the procedure in making an estimate is to ascertain the total cost per linear foot for the superstructure of the main span between towers, omitting the item of back-stays, and apply it to the combined lengths of the main span and side spans. The total costs of the towers, piers, and anchorages are about the same whether the back-stays carry vertical loads or not.

It is true that the stiffening trusses for the side spans are slightly heavier than those for the main span, usually 4 or 5%; but this increase is partly offset by a saving in the weight of the lateral system, because the span is only one-half as long as the main span, and therefore the diagonals are lighter. The longer the spans, the greater will be the proportionate saving from this cause. On the whole, the side spans will be a little more expensive per square foot of floor than the main span. As the cost of the stiffening trusses is generally only about 10% of the total for the superstructure, it is evident that the error caused by assuming equal unit costs for the side spans and the main span is negligible. However, in preparing Fig. 17 cognizance was taken of this slight variation.

These studies aid in determining the extreme feasible lengths of suspension spans. The criterion of feasibility, however, is a mooted point. The

writer in his investigations concerning the use of alloy steels in bridgework* assumed for cantilever structures that the limit of feasibility is reached when it requires 4 lb. extra of metal to support 1 lb. additional live loading. The best possible criterion for a suspension bridge, however, is the financial ability of the interested communities to raise the necessary funds for its construction.

Referring to Fig. 15, it is evident that this limit has not been exceeded even for electric-railway structures with a span of 5 000 ft.; for, with anchorages on pile foundations, the cost per square foot of floor is only $165. As the minimum clear width of roadway for this span is 130 ft., the cost of the bridge proper would be $130 \times 5\,000 \times 165 = \$107\,250\,000$; but to this must be added a large amount for approaches, right of way, property damages, contingencies, engineering, administration, and interest during construction. Judging from the preceding example, the grand total cost would amount to about $150 000 000. Under certain conditions, this amount is not beyond the limit of feasibility, although it is probably outside that of economics for a toll structure, considered as a business venture. For example, if it should require that amount to build a 5 000-ft. span across The Narrows of New York Harbor, and if the adjacent communities in New York and New Jersey were to demand such a structure, the money undoubtedly could be raised.

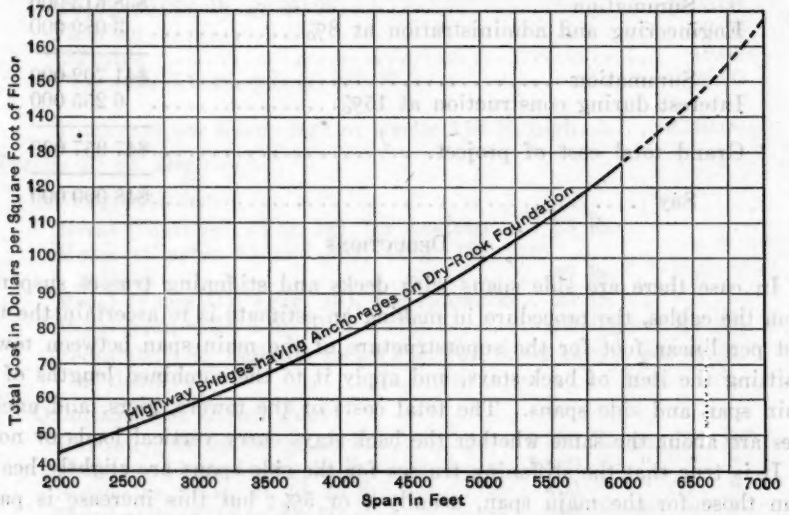

FIG. 21.—TOTAL COST OF LONG-SPAN HIGHWAY SUSPENSION BRIDGES.

Computations of the "approximate greatest lengths of span" have been made† by William H. Burr, M. Am. Soc. C. E., based on certain assumed unit stresses for cables and various ratios of sag to span. His tabulated results are such that, if any additional loading were placed on the structure, the cables would be over-stressed, no matter how much area were added to their cross-section. Using the writer's assumptions, Mr. Burr's tables would indicate an extreme limit of span of about 10 000 ft. This limit satisfies one's

* "Nickel Steel for Bridges," *Transactions*, Am. Soc. C. E., Vol. LXIII (1909), p. 101, and "The Possibilities in Bridge Construction by the Use of High-Alloy Steels," *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 1.

† "Suspension Bridges," p. 74.

curiosity, but does not give information concerning the true or practically feasible limiting span. In the writer's opinion, that limit is reached at about 5 000 ft.; but, in order to show what would occur beyond it, Fig. 21 has been prepared by actually computing the quantities and the costs for a 6 000-ft. span highway bridge, and by extrapolation (as shown by the dotted line) up to a span length of 7 000 ft. For this illustration the lowest line of Fig. 15, representing highway bridges with anchorages resting on bare rock, was chosen as being the most likely condition. From it is made the following estimate of total cost of project for a highway suspension bridge of 7 000-ft. span:

Bridge proper, 1 260 000 sq. ft. @ $166	$209 160 000
Approaches, say, 2 600 ft. @ $600	1 560 000
Retaining walls, say	40 000
Two plazas, say	300 000
Right of way and property damages, say	3 800 000
Summation	$214 860 000
Contingencies, 5%	10 743 000
Summation	$225 603 000
Engineering and administration, 8%	18 048 000
Summation	$243 651 000
Interest during construction $\frac{1}{2} \times 5\%$ for 8 years = 20%	48 730 000
Summation	$292 381 000
Grand total cost of project, say	$300 000 000

This, of course, is a far greater amount than any conditions at present conceivable would warrant.

A similar calculation for a 6 000-ft. span makes the grand total cost of project nearly $180 000 000, which probably is also too great for practical consideration.

Figs. 16 to 20, inclusive, make possible a comparison of the costs of suspension bridges with and without suspended side spans. Compare the case already computed, namely, a 3 800-ft. span, assuming 1 900-ft. approaches, with a similar bridge that has suspended side spans of 1 900 ft. each. The deck, the main piers, the anchorages, the tower encasement, and the lighting will cost nearly the same in both cases, and hence may be ignored.

Cost of Metal

One-Span Structure.—

Structural steel, highway (Fig. 16)	240 lb.	@ $0.094 (ave.)	$\times$ 70 ft.	= $1 579.20	per lin. ft.
Cables, highway (Fig. 16)	216 "	@ 0.16	$\times$ 70 ft.	= 2 419.20	" " "
Structural steel, railway (Fig. 16)	330 "	@ 0.094 (ave.)	$\times$ 40 ft.	= 1 240.80	" " "
Cables, railway (Fig. 16)	284 "	@ 0.16	$\times$ 40 ft.	= 1 817.60	" " "
Total				$7 056.80	" " "

Total cost of metal = 3 800 $\times$ $7 056.80 = $26 816 000

Trestle Approaches for One-Span Structure.—

Metal	10 000 lb. @ $0.06	= $600	per lin. ft.
Pedestals	1.2 cu. yd. @ $20	= 24	" " "
Total		= $624	" " "

Total cost of trestle = 3 800 × $624 = $2 371 000

Three-Span Structure.—

Structural steel, highway (Fig. 17)	190 lb. @ $0.094 (ave.)	× 70 ft.	= $1 250.00	per lin. ft.
Cables, highway (Fig. 17)	110 " @ 0.16	× 70 ft.	= 1 232.00	" " "
Structural steel, railway (Fig. 17)	258 " @ 0.094 (ave.)	× 40 ft.	= 970.10	" " "
Cables, railway (Fig. 17)	140 " @ 0.16	× 40 ft.	= 896.00	" " "
Total			$4 348.30	" " "

Total cost of metal = 7 600 × $4 348.30 = $33 047 000

Comparison of Costs

Cost of one-span bridge	$26 816 000
Cost of approaches	2 371 000
Comparing cost for one-span structure	$29 187 000
" " " three-span "	33 047 000
Difference in favor of one-span structure	$ 3 860 000

The total cost of the one-span structure would be:

Main structure, 3 800 ft. @ $8 375	$31 825 000
Approaches, 3 800 ft. @ $740	2 812 000
Retaining walls	34 000
Total comparing cost	$34 671 000

Percentage of increase for three-span structure:

$$3\,860\,000 \times 100 \div 34\,671\,000 = 11, \text{ nearly.}$$

This is good evidence of the correctness of a claim made by the writer many years ago, namely, that there is no economy in adopting suspended side spans for suspension bridges when steel trestles or even short, independent spans can be used instead.

Conclusion

The computations for this paper have been carefully checked, hence the results are reliable. Nevertheless, owing to the meager treatment to date of the design of long-span suspension bridges, it is probable that another engineer in making a similar investigation would obtain somewhat different results, even if he were to adhere to the same specifications for design.

It is hoped that many members of the Society will discuss these questions from all angles, and will bring up and treat important side-lights, thus adding to the general knowledge of the Engineering Profession concerning long-span suspension bridges.

VENTURI TUBE CHARACTERISTICS

By J. W. Ledoux,* M. Am. Soc. C. E.

Synopsis

This paper discusses the Venturi tube and gives the coefficients of discharge and losses of head determined from a series of actual tests, made under the direction of the writer, of nineteen tubes ranging in size from 30 in. to 1 in. There is also given in the paper results of tests of eight tubes, obtained by other experimenters.

The effect of various angles of convergence and divergence, tube ratios, and size of tubes on the coefficients of discharge and losses of head, is discussed; and the conclusion is reached that for Venturi tubes ranging in size from 4 to 30 in., the coefficient of discharge is approximately the same, and ranges from 0.96 to 0.985, with an average of about 0.977 for high velocities, and from 0.90 to 0.94 for low velocities.

From the results of the tests of the twenty-four tubes, the following formula for loss of head is deduced:

$$f = \frac{V_1^{1.96}}{403}$$

which is roughly expressed as:

$$f = \frac{V_1^2}{470}$$

which is stated in another way as,

$$f = 0.13\, H \left(\frac{R^4}{R^4 - 1}\right)$$

The opinion is also given that the Venturi tube is only surpassed in accuracy by actual weighing or volumetric measurements. While a V-notch or rectangular weir is an excellent device for approximate measurements, it is not sufficiently accurate for use as a basis for checking the accuracy of a well-designed Venturi tube.

Every hydraulic engineer is familiar with the main characteristics of the flow of water through orifices and Venturi tubes. The advantage of the Venturi tube over an orifice is that for an equal differential head the unrecoverable loss of head for the orifice is much greater than with the Venturi tube.

Note.—Written discussion on this paper will be closed in February, 1927. When finally closed the paper, with discussion in full, will be published in *Transactions*.

* Cons. Engr., Simplex Valve & Meter Co., Philadelphia, Pa.

It was formerly believed that Venturi tubes, to be accurate, should be made with definite and invariable lines; that the ratio of the main diameter to that of the throat should be fixed, and that the angles of divergence and convergence should always be the same. Within the past ten years, exhaustive tests have been made, which show that such requirements are unnecessary, and that the ratios as well as the angles of divergence may vary through a wide range without materially affecting the coefficient of discharge.

The most convenient formula for the Venturi tube is as follows:

$$V = c\left(\frac{2\,g\,H}{R^4 - 1}\right)^{\frac{1}{2}}$$

in which,

V = velocity through the main, or D section, in feet per second.
H = the Venturi head, in feet of water.
R = the ratio of main diameter, D, to throat diameter, d.
g = the acceleration of gravity.
c = the coefficient of discharge through the Venturi tube.

The unrecoverable loss of head through a clean Venturi tube may be expressed approximately by the formula:

$$f = \frac{V_1^2}{470}$$

in which,

f = the total loss of head across the Venturi tube, measured in feet.
V_1 = the velocity through the throat, in feet per second.

The exponent of V_1 is designated n in subsequent references. For,

$$c = 0.977,$$

and since,

$$V^2 = \frac{V_1^2}{R^4}$$

$$f = 0.13\,H\left(\frac{R^4}{R^4 - 1}\right)$$

It must be understood that these approximate formulas will not apply invariably to all tubes, as the tendency is toward larger values of f for smaller throat diameters.

It was also thought at one time that for a given velocity through the throat the loss of head was greater for the higher ratio of diameters. Actual tests do not confirm this with certainty. As to the angle of divergence of the downstream end of the Venturi tube, in many tubes this is 2½°; in many others, it is 6½°, but the effect on the coefficient is imperceptible.

With a given velocity through the throat it cannot be stated with certainty that the loss of head, f, is proportional or inversely proportional to this angle; neither does the loss of head vary exactly as the square of the velocity through the throat. The average of all the results available, including the writer's and

those of other experimenters, shows the exponent, n, to be 1.961, the average formula being:

$$f = \frac{V_1^{1.961}}{403}$$

It was formerly taken for granted that coefficients of Venturi tubes varied with the size of the tube, the higher coefficients being obtained with the larger diameters. While that is probably true for tubes more than 4 in. in diameter, as compared with those less than 2 in. in diameter, there does not seem to be a great difference between the tubes as small as 4 in. in diameter and those as great as 24 in., or greater. William S. Pardoe, M. Am. Soc. C. E., has developed a formula which indicates that coefficients are higher with large tubes than with small ones, but the results of tests do not establish conclusively the invariable reliability of such a formula.

In making tests for coefficients of Venturi tubes, it is necessary to measure the quantity of water with great accuracy, and as the Venturi tube is such an accurate device itself, it cannot be checked with any less accurate apparatus. The only devices that are more accurate are tanks and weighing machines. All the tests herein referred to for coefficients, on which conclusions are based, were made on the weighing apparatus at the University of Pennsylvania, or the tank apparatus at Cornell University. For loss of head, however, the data obtained by weirs are near enough to warrant retaining the results, but for coefficients, weirs are not sufficiently accurate.

The measurement of the Venturi head also requires the greatest accuracy. Formerly, these heads were measured by manometers, but it was found that a much more reliable plan was to use hook-gauges, determining the actual levels to which the water rises in the main section of the tube and in the throat section, and taking the difference between those levels as the Venturi head. Such measurements of head are correct to 0.02 in., whereas with manometers, errors of 0.1 in. are common and almost unavoidable.

It was once believed that the coefficients of discharge were greater with low than with high throat velocities, but this opinion has been disproved by overwhelming evidence, and every tube tested under the writer's direction has shown the coefficient to increase rapidly with the increase of throat velocity from zero up to 8 or 10 ft. per sec.

To show the findings from various previous tests for loss of head, Table 1 has been compiled. The results of all the tests, except No. 7, followed an exponential formula; that is, they conform almost exactly to a straight line on logarithmic paper. The Coker and Strickland results, however, showed a considerable variation in the middle part of the series.

Opposite each test are given values of the constants a and n in the formula for the loss of head in the form:

$$f = \frac{V^n_1}{a}$$

To deduce the values of the constants, a and n, the actual losses of head and throat velocities were plotted on logarithmic paper, and an equation was written for the straight line passing through the points representing loss of head.

TABLE 1.—FORMER TESTS FOR LOSS OF HEAD.

Test number.	Ratio of diameters.	Diameter of main, in inches.	Diameter of throat, in inches.	Angle of convergence, in degrees.	Angle of divergence, in degrees.	Losses of Head, f, in Feet, for Various Throat Velocities.							Values of Constants in Formula for Loss of Head, $f = \frac{V_1^n}{a}$.	
						10 ft.	15 ft.	20 ft.	25 ft.	30 ft.	35 ft.	40 ft.	a.	n.
1	3	108	36	10½	2½	0.19	0.39	0.70	1.08	1.53	2.18*	2.70*	477	1.94
2	2.11	54	25.5	10½	2½	0.23	0.53	0.94	1.43*	2.10*	2.85*	3.77	458	2.02
3	2.13	48	22.5	10½	2½	0.16	0.34	0.62	1.00	1.45*	1.98*	2.60*	698	2.03
4	3	48	16	10½	2½	0.15	0.34	0.62	0.98	1.43	1.95*	2.60*	804	2.07
5	3	12	4	10½	2½	0.26	0.56	0.97	1.45	2.10	2.80*	3.65*	303	1.90
6	2.33	12	5.16	10½	2½	0.25	0.55	0.94	1.47	2.10	2.68*	3.65	355	1.94
7	4.27	1.625	0.38	10½	2½	0.29	0.89	1.50	2.10	2.80	3.50	4.40	340	1.99

NOTE.—Test No. 1 made by Herschel, at Holyoke Flume, *Transactions*, Am. Soc. C. E., Vol. XVII (1887), p. 228.
" " 2 " " Marx, Wing and Hoskins, *Transactions*, Am. Soc. C. E., Vol. XL (1898), p. 471.
" " 3 " " Herschel, using weir.
" " 4 " " " " "
" " 5 " " " " tank.
" " 6 " at Cornell University, using tank.
" " 7 " by Coker and Strickland, *Transactions*, Eng. Inst. of Canada, 1902; also, "Hydraulics," by Hughes and Safford, 1912, p. 118.

*Loss of head figures interpolated on logarithmic paper by the writer

TABLE 2.—LEDOUX TESTS FOR VALUES OF c, f, a, AND n.

Test number.	Ratio of diameter.	Diameter of main, in inches.	Diameter of throat, in inches.	Angle of convergence, in degrees.	Angle of divergence, in degrees.	Factors.	VALUES OF COEFFICIENT, c; AND LOST HEAD, f, IN FEET, FOR VARIOUS THROAT VELOCITIES. 1 ft.	2 ft.	4 ft.	6 ft.	8 ft.	10 ft.	15 ft.	20 ft.	25 ft.	30 ft.	35 ft.	40 ft.	VALUES OF CONSTANTS IN FORMULA FOR LOSS OF HEAD, $f = \frac{V_1^n}{a}$. a.	n.
8	1.5	24	16	10½	6½	c	0.930	0.952	0.966	0.973	0.976	0.977	0.977	0.976	0.975	0.973	0.970	0.969	242	1.67
						f	.....	.....	.....	.....	.....	0.20	0.38	0.63	0.92	1.25	1.62	1.97*	....	....
9	1.67	20	12	10½	6½	c	.....	0.968	0.977	0.979	0.980	0.980	0.981	0.982	0.982	0.982	.....	.....	846	2.14
						f	.....	.....	.....	.....	.....	0.16	0.40	0.78	1.16	1.75	2.40	3.15*	....	....
10	2	16	8	10½	2½	c	0.935	0.947	0.960	0.968	0.971	0.972	.....	.....	.....	.....	.....	.....	.....	.....
11	2	12	6	10½	6½	c	0.912	0.949	0.965	0.970	0.973	0.974	0.975	0.976	0.976	0.975	.....	.....	353	1.86
						f	.....	.....	.....	.....	.....	0.20	0.44	0.77	1.14	1.52	2.10*	2.70*	....	....
12	1.47	6.02	4.01	10½	6½	c	0.951	0.964	0.976	0.985	0.990	0.991	0.989	0.989	0.989	0.989	0.989	.....	636	2.03
						f	.....	.....	.....	.....	.....	0.19	0.39	0.71	1.11	1.57	2.16	2.85*	....	....
13	2.08	7.95	3.8	10½	2½	c	0.930	0.970	0.978	0.981	0.981	0.981	0.982	0.984	0.987	0.989	0.990	.....	....	....
14	1.995	6.04	3.028	10½	6½	c	0.922	0.942	0.956	0.966	0.973	0.975	0.975	0.975	0.975	0.975	.....	.....	237	1.87
						f	.....	.....	.....	.....	.....	0.32	0.65	1.13	1.76	2.42	3.04	4.12	....	....
15	2	6	3	10½	2½	c	0.930	0.950	0.968	0.973	0.975	0.975	0.976	0.976	0.976	0.976	.....	.....	388	1.93
						f	.....	.....	.....	.....	.....	0.22	0.48	0.83	1.30	1.82	2.47*	3.20*	....	....
16	2.24	6.08	2.72	10½	6½	c	0.910	0.931	0.950	0.960	0.966	0.969	0.969	0.969	0.969	0.969	.....	.....	....	....
						f	.....	.....	.....	.....	.....	0.40	1.0	1.85	2.90	4.10	5.55	.....	....	....
17	3	8	2.67	10½	2½	c	0.930	0.960	0.969	0.974	0.977	0.978	0.981	0.982	0.982	0.982	.....	.....	603	2.08
						f	.....	.....	.....	.....	.....	0.20	0.45	0.87	1.33	1.87	2.77	3.58*	....	....
18	2.98	6.02	2.02	10½	6½	c	0.895	0.908	0.923	0.933	0.942	0.946	0.947	0.946	0.945	0.945	0.945	0.945	235	1.98
						f	.....	.....	.....	.....	.....	0.40	0.90	1.60	2.50	3.50	4.77	6.20*	....	....
19	1.50	3	2	10½	6½	c	.....	.....	.....	.....	.....	0.963	0.966	0.965	0.962	0.962	9.66	.....	248	1.79
						f	.....	.....	.....	.....	.....	0.25	0.46	0.77	1.31	1.77	2.34	3.00*	....	....
20	1.99	3.97	2	10½	2½	c	0.930	0.945	0.956	0.963	0.966	0.968	0.970	0.970	0.970	0.970	0.970	.....	306	1.92
						f	.....	.....	.....	.....	.....	0.27	0.59	1.08	1.55	2.20	2.97	3.85†	....	....
21	1.97	3.928	2	10½	2½	c	0.925	0.941	0.955	0.963	0.968	0.972	0.973	0.975	0.976	0.976	.....	.....	....	....
22	2.61	4	1.532	10½	6½	c	0.900	0.922	0.945	0.954	0.959	0.962	0.966	0.969	0.970	.....	.....	.....	310	2.07
						f	.....	.....	.....	.....	.....	0.40	0.88	1.53	2.55	3.75†	5.10†	6.70†	....	....
23	2.01	2.01	1	10½	2½	c	0.928	0.936	0.945	0.951	0.956	0.960	0.963	0.965	0.966	.....	.....	.....	181	1.75
						f	.....	.....	.....	.....	.....	0.28	0.60	1.10	1.52	1.97	2.70†	3.45†	....	....
24	2	1	0.5	10½	6½	c	0.900	0.912	0.932	0.942	0.947	0.951	0.957	0.960	.....	.....	.....	.....	....	....
25	1.53	0.625	0.409	10½	6½	c	0.880	0.901	0.932	0.946	0.953	0.955	0.959	0.960	.....	.....	.....	.....	....	....
26	Averages of all the coefficients, (Tests Nos. 8 to 25)					c	0.919	0.941	0.957	0.964	0.968	0.969	.....	0.971	0.973	0.974	.....	.....	....	....
27	"	omitting Tests Nos. 24 and 25				c	0.923	0.946	0.960	0.967	0.971	0.971	.....	0.973	0.973	0.974	.....	.....	....	....
28	"	of Tests Nos. 8, 9, 10, and 11				c	0.926	0.954	0.968	0.973	0.975	0.976	.....	0.978	0.978	0.977	.....	.....	....	....
29	"	of Tests Nos. 8, 9, and 10				c	0.932	0.956	0.968	0.974	0.976	0.976	.....	0.979	0.978	0.978	.....	.....	....	....
30	"	Tests Nos. 8 and 9				c	0.930	0.960	0.972	0.976	0.978	0.978	.....	0.979	0.978	0.977	.....	.....	....	....

*Results interpolated.

†Loss of head figures interpolated on logarithmic paper by the writer.

For coefficients, only those results were used wherein the tests were made under the writer's direction, so that he can vouch for them. In a few exceptional instances, coefficients at high velocities were obtained, reaching from 0.99 to unity for 6 and 8-in. tubes. In no instance were coefficients found high at low velocities and low at high velocities. The writer's tests of tubes made by other manufacturers, in all their characteristics, agree substantially with those of his own design and make.

In a few cases, where the writer's Venturi tubes showed coefficients of 0.99 up to more than unity, it was concluded that the results for some reason were unduly high. Assuming that this might be caused by an incorrect Venturi head, due to vortex action at the up-stream section, straightening vanes were inserted, and immediately the results came down to normal. This is a further reason for skepticism concerning coefficients greater than about 0.98.

In Table 2 are given the results of various tests made under the writer's direction. Tests Nos. 8 and 9 were made at Cornell University by E. W. Schoder, M. Am. Soc. C. E., of Cornell, and Professor Pardoe, of the University of Pennsylvania. All the others were made at the Hydraulic Laboratory of the University of Pennsylvania by Professor Pardoe. Tests Nos. 20 and 21 were made with tubes manufactured by the Builders Iron Foundry. Test No. 16 was made with a Venturi tube having its invert in a single straight line, the angles of convergence and divergence being at the top, 21° and 13°, respectively, and the cross-sections being circular throughout.

TABLE 3.—TESTS FOR ACCURACY OF SIMPLEX VENTURI TUBES.

Item.	Throat Velocities, in Feet per Second.								
	1.	2.	4.	6.	8.	10.	20.	25.	30.
(*a*) SIMPLEX ADOPTED COEFFICIENTS									
Coefficient, *c*.	0.934	0.949	0.964	0.969	0.972	0.972	0.976	0.977	0.977
(*b*) PERCENTAGE OF ERROR OF SIMPLEX COEFFICIENTS									
Based on average of all tests, Series 26	+1.64	+0.85	+0.78	+0.52	+0.41	+0.31	+0.51	+0.41	+0.31
Based on Series 27	+1.18	+0.32	+0.42	+0.21	+0.10	+0.10	+0.31	+0.41	+0.31
Based on Series 28	+0.86	−0.52	−0.41	−0.41	−0.31	−0.41	−0.20	−0.10	0
Based on Series 29	+0.21	−0.73	−0.41	−0.51	−0.41	−0.41	−0.31	−0.10	−0.10
Based on Series 30	+0.43	−1.15	−0.82	−0.72	−0.61	−0.61	−0.31	−0.10	0
Based on Test No. 9	.....	−1.96	−1.33	−1.02	−0.82	−0.82	−0.61	−0.51	−0.51
Based on Test No. 12	−1.79	−1.56	−1.23	−1.63	−1.82	−1.92	−1.31	−1.21	−1.21
Based on Test No. 18	±0.43	−2.17	−1.43	−1.23	−0.92	−0.92	−0.81	−1.01	−1.21

The Venturi meter register designed by the writer was made in 1912 to conform with the results of tests of three tubes, 16, 8, and 4-in., with a ratio, R, of approximately 2, and half angles of $10\frac{1}{2}°$ up stream and $2\frac{1}{2}°$ down stream, in each case. No change in the design of this vital part has been made since that date; the subsequent tests have verified and confirmed the original design.

In Table 3 are given percentages of error which are conclusive evidence that the coefficients adopted were justified. In the tests of Table 3 the nominal and

not the exact diameter of the Venturi tubes has usually been given, but in plotting the coefficients the exact diameter was used.

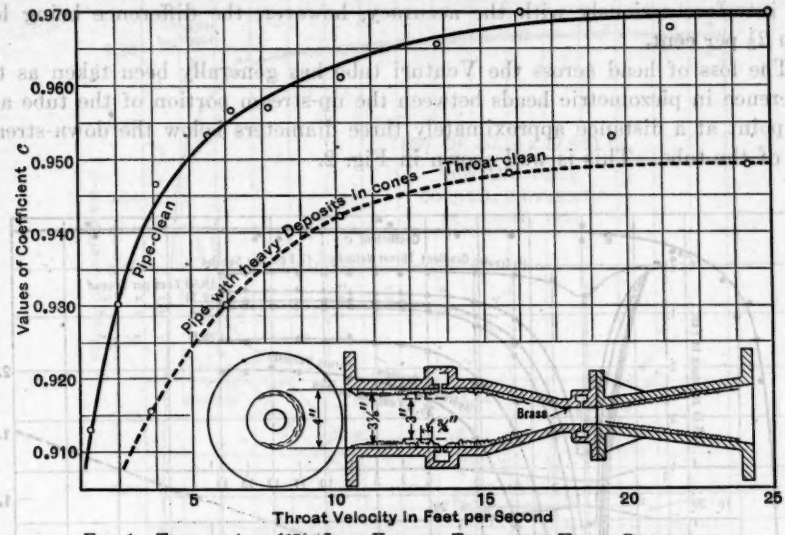

FIG. 1.—TEST OF 4 BY 1 17/32-INCH VENTURI TUBE, WITH HEAVY DEPOSIT IN CONES, THROAT CLEAN.

Test No. 22 (Table 2) was made with a clean tube. This tube had been considerably tuberculated, and a test of it was also made before cleaning. The tests gave the coefficients shown in Table 3 and in Fig. 1.

TABLE 4.—TESTS SHOWING EFFECT OF CLEANING VENTURI TUBES, ON THE COEFFICIENT OF DISCHARGE, *c*.

Run.	*H*, in feet.	Theoretical velocity.	Flow, in cubic feet per second.	V_1, in feet per second.	*c*.	*f*, in feet, per tube plus 18 in. of 4-in. pipe.
			(*a*) TUBE WITH HEAVY DEPOSITS IN CONES			
1.....	0.2519	4.07	0.004775	3.73	0.916	0.296
2.....	0.6844	6.71	0.0798	6.24	0.980	0.850
3.....	1.770	10.77	0.01301	10.16	0.942	2.240
4.....	4.294	16.78	0.2086	15.92	0.948	5.450
5.....	6.380	20.47	0.2497	19.52	0.953	8.150
6.....	9.904	25.50	0.3097	24.18	0.949	12.800
			(*b*) TUBE CLEAN			
1.....	0.04955	1.804	0.0211	1.648	0.913	0.0166
2.....	0.11115	2.703	0.03218	2.515	0.930	0.030
3.....	0.25111	4.065	0.04925	3.845	0.9465	0.070
4.....	0.58572	5.935	0.0724	5.660	0.953	0.148
5.....	0.6976	6.770	0.0830	6.480	0.9565	0.178
6.....	1.005	8.130	0.09965	7.780	0.957	0.302
7.....	1.733	10.67	0.1312	10.25	0.961	0.406
8.....	2.997	14.03	0.1736	13.56	0.966	0.730
9.....	4.345	16.89	0.2100	16.41	0.970	1.045
10.....	7.550	22.26	0.2760	21.55	0.968	1.760
11.....	10.065	25.73	0.3197	24.98	0.970	2.520

The loss of head in the dirty tube for a 25½-ft. velocity through the throat was 12.8 ft., and for the clean tube, 2.5 ft. This enormous difference did not interfere seriously with the accuracy, however, the difference being less than 2½ per cent.

The loss of head across the Venturi tube has generally been taken as the difference in piezometric heads between the up-stream portion of the tube and the point at a distance approximately three diameters below the down-stream end of the tube. This is well shown in Fig. 2.

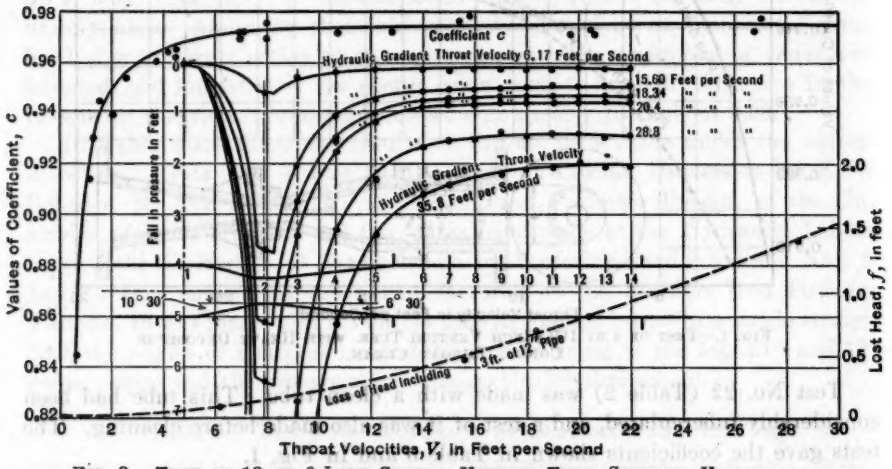
Coefficient c
Hydraulic Gradient Throat Velocity 6.17 Feet per Second
15.60 Feet per Second
18.34 " " "
20.4 " " "
28.8 " " "
Hydraulic Gradient Throat Velocity 35.8 Feet per Second
10° 30'
6° 30'
3 ft. of 12" Pipe
Loss of Head including
Fall in pressure in Feet
Values of Coefficient, c
Lost Head, f, in feet
Throat Velocities, V, in Feet per Second

FIG. 2.—TEST OF 12 BY 6-INCH SPECIAL VENTURI TUBE, SHOWING VALUES OF c, HYDRAULIC GRADE LINES, AND LOSSES OF HEAD.

In addition to those tests already noted, one on a 30 by 18-in. tube was made under the writer's direction at Cornell University (February, 1926), by Professors Schoder and Pardoe. The capacity—about 58 cu. ft. per sec.—was the extreme limit of that Laboratory. The coefficients, although somewhat high for the high throat velocities, are in close agreement with those of previous tests. As this is the largest tube tested under the writer's direction at a university laboratory, the results are reproduced herewith (Fig. 3).

The coefficients for throat velocities of more than 5 ft. appear to be somewhat high, which may be caused by slight errors in the observed Venturi head due to vortex action at the up-stream point of the Venturi tube. The arrangement of the piping, as shown in Fig. 3, would lend color to this supposition. Possibly if straightening vanes had been inserted above the Venturi tube, the coefficients might have been somewhat reduced. However, the coefficients, except at low velocities, follow closely the curve computed by Professor Pardoe.

The conditions for making the loss of head measurements were not favorable on account of the necessity of discharging from a 30-in. into a 36-in. pipe (not shown in Fig. 3), so that the recovery of head was impaired. The results, as given in Fig. 3 and Table 5, have been corrected by calculation, on the assumption that there were three diameters (90 in.) of 30-in. pipe below the down-stream end of the Venturi tube.

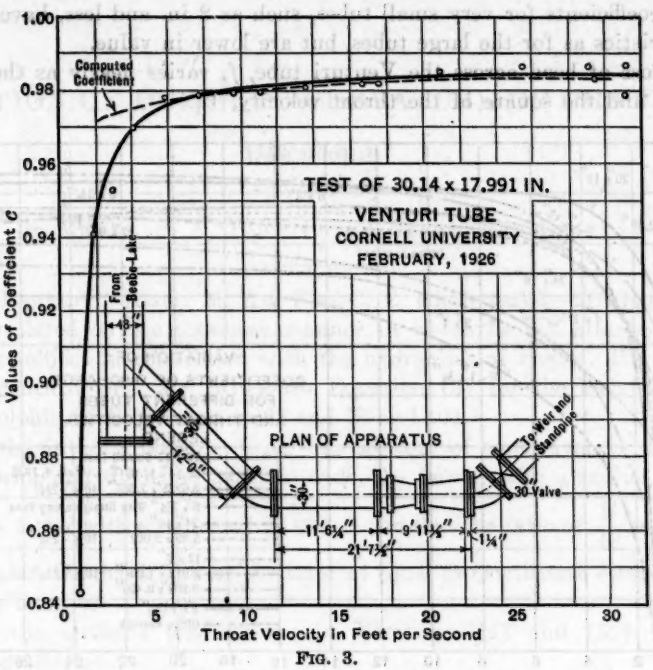

FIG. 3.

Fig. 4 shows typical curves of the coefficients for different sized Venturi tubes. It will be seen that for velocities more than about 16 ft. through the throat, the coefficients vary from 0.96 to 0.985.

TABLE 5.—TEST OF 30 BY 18-INCH TUBE.

(Test No. 25A, February, 1926; Ratio of Diameters, $R = 1.67$; Angles of Convergence and Divergence, $10\frac{1}{2}$ and $6\frac{1}{2}$ Degrees.)

Factors.	Throat Velocities, in Feet per Second.										
	1.	2.	4.	6.	8.	10.	15.	20.	25.	30.	35.
Value of coefficient, c	0.85	0.946	0.968	0.976	0.979	0.980	0.982	0.983	0.984	0.984	0.984
Loss of head, f	.....	.....	.....	.....	.....	0.34	0.73	1.10	1.48	1.83	2.26

The salient conclusions are that for throat velocities, V_1, of more than 4 ft. per sec., the coefficients range between 0.96 and 0.985 for Venturi tubes more than 8 in. in diameter. For throat velocities of more than 8 ft. per sec., the coefficients are almost constant, and for throat velocities of less than 4 ft. per sec., the coefficients fall off rapidly. If due correction is made for the falling off of the coefficients at low velocities, the Venturi tube is an extremely accurate measuring device.

The coefficients for very small tubes, such as 2 in. and less, have the same characteristics as for the large tubes, but are lower in value.

The loss of head across the Venturi tube, f, varies nearly as the Venturi head, H, and the square of the throat velocity, V_1.

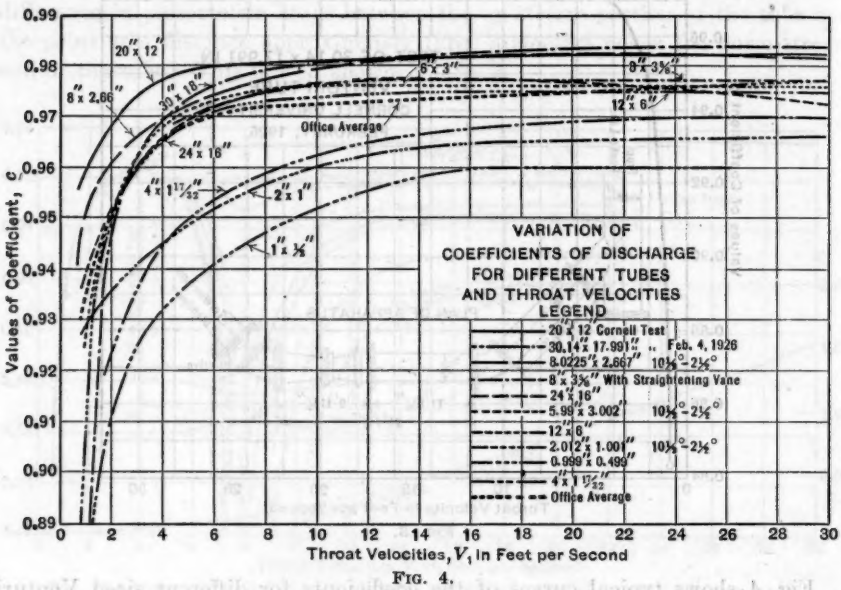

FIG. 4.

Some of the early opinions, to the effect that the Venturi tube coefficient decreases as the velocity increases, and is a maximum at the lowest velocity, have been completely disproved, the reverse being the case.

ACKNOWLEDGMENTS

Acknowledgment is due W. S. Pardoe, M. Am. Soc. C. E., under whose immediate direction the experiments at the Laboratory of the University of Pennsylvania were carried out. Acknowledgment is also due E. W. Schoder, M. Am. Soc. C. E., who, assisted by Professor Pardoe, carried out the experiments at the Hydraulic Laboratory, Cornell University, referred to in Tables 2 and 5.

RELATION OF DEPTH TO CURVATURE OF CHANNELS

Discussion*

By Messrs. P. N. Fawcett and Henry Clay Ripley.†

P. N. Fawcett,‡ Assoc. M. Am. Soc. C. E. (by letter).§—Mr. Ripley is to be congratulated on the thorough manner in which he has attacked a most important subject in connection with the hydraulics of rivers. His Formula (4)‖ is identical with Mr. Mitchell's Equation (3) for the Delaware River at Philadelphia, if D be taken as 33 and W as 1 100.

With the author's formula, using two sections of equal average depth, one 4 000 ft. wide and the other 2 000 ft. wide, the former with a radius of 40 000 ft. and the latter with a radius of 20 000 ft., so that the ratio of $\frac{R}{W}$ is the same in both cases, the depth will be the same at equal proportionate distances from the center line.

From the author's assumptions in Formulas (4) and (5)‖ in which $\frac{P}{D} = \frac{1.65}{1.445}$, and the constant in the second part of the formula is in the proportion, $\frac{26.28}{17.52}$, it naturally follows that any depth calculated by Formula (5) with one training wall will be greater than the depths calculated from Formula (4) with two training walls. Reverse the formulas and the single training wall will produce by calculation a less depth than the double training walls. This does not prove, however, that such results will follow, as stated by the author.

It is obvious that with a greater constant, greater depths will be obtained by calculations and *vice versa*, but there has been no allowance made in the formula for the "greater facility with which the material excavated is dispersed" so that it is difficult to understand how greater depths are claimed to be obtained from this formula on this account.

As regards the statement that a single training wall does not cause a bar advance, such a statement would appear to be contrary to reason. The material eroded from within the bar must be carried seaward whether the confinement of the currents causing the erosion is obtained by one training wall or two walls, and this material (other things, such as littoral current, being the same in both cases) must cause a bar advance.

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* Discussion on the paper by H. C. Ripley, M. Am. Soc. C. E., continued from October, 1926, *Proceedings*.

† Author's closure.

‡ Engr.-in-Chf., Lower Liao River Conservancy, Newchwang, Manchuria, North China.

§ Received by the Secretary, August 20, 1926.

‖ *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1909.

Cases in which bars obstructing the mouths of rivers are being trained by one training wall only, unassisted by dredging, are extremely rare. In fact, the writer is unable to quote any cases. Under these circumstances doubtless some of the particulars of the bar of the Liao River, in Manchuria, will be of interest.

The bar of the Liao River is being trained by one training wall constructed first, after which it will be seen whether a second wall will be required. It is highly probable that a second training wall will never be constructed, reliance being placed on one wall, assisted by dredging, if necessary. At present (July, 1926), this wall has been built to a length of 35 000 ft. It still requires a further 6 000 ft. to be constructed. Its top is about 3 ft. above low-water level only, as any further height requires heavy block stone on account of the ice effect during the winter months.

This wall is producing a considerable improvement as will be seen from Fig. 13. It should be noted, however, that the bar is advancing the 9-ft. sea contour, which has been moved seaward 3 000 ft. since 1917 and other contours *pro rata*. Although this advance is not serious, it is nevertheless a fact and disproves Mr. Ripley's theory that a single training wall does not cause a bar advance.

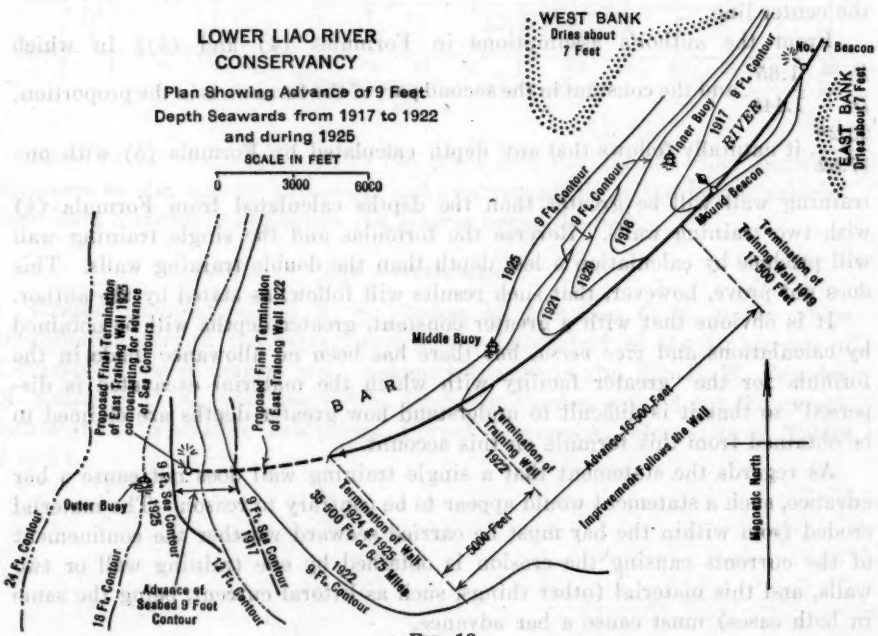

FIG. 13.

A few particulars of this port will be of advantage in considering the situation. Spring tides rise 13 ft. Flood and ebb currents attain about 3½ knots across the bar and up to 4½ knots in the river proper. The ebb runs

across the wall S. 40 W. to S. 60 W. and the flood in a direction N. 34 E. to N. 40 E. The prevailing winds are northeast during the winter months while the port is closed by ice, and southwest during the summer months.

Thus it will be seen that the ebb tide hugs the training wall throughout its entire length, there being full play for centrifugal force, so that the situation is entirely favorable to the success of a single wall. The soil is the light loess of the North China plains, so fine that 50% will pass through a sieve of 144 meshes per sq. cm. When compacted on the bed of the bar it forms a hard impenetrable surface and becomes extremely difficult to erode or to dredge by suction dredges.

HENRY CLAY RIPLEY,* M. AM. SOC. C. E. (by letter).†—The writer wishes to express his appreciation of the many helpful criticisms of his paper and he is especially pleased with the numerous responses from foreign countries. England, France, Germany, and China have contributed to this discussion.

The interest shown by Mr. Walker,‡ although it is in the mathematical feature only, is gratifying. While recognizing that Equation (8)‡ is more concise than Formulas (4) and (5)§, it is considered that what is gained in conciseness is lost in ease of solution. The value of these formulas is for computing the co-ordinates of the cross-profile of a given channel or its maximum depth by the use of Equations (*a*) or (*b*).‖ Equation (8) in its present form cannot be used for these purposes for it does not contain the elements necessary to those ends, as Y and X in Equation (8) are not the co-ordinates of the curve to be reproduced and it does not contain D, W, and R, all of which are in immediate demand when one undertakes to compute the co-ordinates of the curve.

However, the writer quite agrees with Mr. Walker in the possibility of modifying the formulas so as to get closer approximations to the actual curves. Since submitting his paper, he has given some further study to this feature of the subject.

The basis for the formulas submitted is that of a conic section and the first term of the second member of Formulas (4) and (5) is that of a parabola the focal distance of which from the apex is equal to $\frac{W^2}{4\,D}$, and in Formulas (5) and (7)§ this distance is equal to $\frac{W^2}{4\,P}$ and with the origin at a point on the axis at a distance from the apex equal to D and P, respectively.

In a straight channel, the equation of the arc of a circle the chord of which is 2 W, the middle ordinate, D, and the radius, $\frac{D^2 + W^2}{2\,D}$, gives almost iden-

* Cons. Engr., Detroit, Mich.

† Received by the Secretary, July 30, 1926.

‡ *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 532.

§ *Loc. cit.*, December, 1925, Papers and Discussions, p. 1909.

‖ *Loc. cit.*, p. 1937.

tical results with Formula (4). The equation of a circle with these conditions is:

$$Y = \pm \sqrt{\left(\frac{D^2 + W^2}{2\,D}\right) - X^2} - \frac{D^2 + W^2}{2\,D} + D \ldots\ldots\ldots\ldots (11)$$

Both Equation (11) and Formula (4) give the area of the cross-section about 4% less than the actual area. With the ellipse, however, an equation may be obtained which will give a computed area the same as the actual area and, at the same time, a computed cross-profile more closely approximating the actual. This equation is determined as follows.

The standard equation of the ellipse the semi-transverse axis of which is a, and the semi-conjugate axis, b, with the origin at their intersection, is:

$$x^2\,b^2 + y^2\,a^2 = a^2\,b^2$$

With the origin removed to a point distant from the center equal to $-k$, and the X-axis parallel to the transverse axis and the Y-axis passing through the center,

$$x^2\,b^2 + (y - k)^2\,a^2 = a^2\,b^2$$

In this case, $b^2 = (D + k)^2$ and $a^2 = W^2\left(1 + \frac{k^2}{D^2 + 2\,D\,k}\right)$, for when $y = 0$, $x = W$. Substituting these values of a^2 and b^2 and reducing,

$$y = \pm \sqrt{(D^2 + 2\,D\,k)\left(1 - \frac{x^2}{W^2}\right) + k^2 + k}$$

By making $k = 1.9\,D$,

$$y = \pm\, D \sqrt{\left(4.8\left(1 - \frac{x^2}{W^2}\right) + 3.61 - 1.9\right)}$$

Limiting the equation to that part of the ellipse below the X-axis, giving y a positive value, and providing for curvature, the complete equation becomes:

$$y = D \sqrt{\left(4.8\left(1 - \frac{x^2}{W^2}\right) + 3.61 - 1.9\right)} + D\,\frac{17.52}{R} \sqrt{\left(4.8\left(1 - \frac{x^2}{W^2}\right) + 3.61 - 1.9\right)}\,x \ldots\ldots\ldots\ldots (12)$$

For measurements in meters,

$$y = D \sqrt{\left(4.8\left(1 - \frac{x^2}{W^2}\right) + 3.61 - 1.9\right)} + D\,\frac{5.34}{R} \sqrt{\left(4.8\left(1 - \frac{x^2}{W^2}\right) + 3.61 - 1.9\right)}\,x \ldots\ldots\ldots\ldots (13)$$

Equations (12) and (13) give the area of the cross-section exactly and the cross-profile a little nearer to the actual than Formulas (4) and (5). It cannot be doubted that subsequent investigators will be able to devise an equation that will give a still closer approximation to the actual cross-profile. This is a fruitful field for future study. The writer believes, however, that the formulas in their present shape, together with the notations, will give results sufficient

for all practical purposes and will enable the engineer to design his works with a confidence in their success never heretofore experienced.

Table 30 gives the co-ordinates of the cross-profile of the Brazos River in a straight reach computed from Formula (4), a parabolic curve, from Equation (11), the arc of a circle, and from Equation (12), the curve of an ellipse (see Table 1* and Fig. 1 (*A*)),† together with the co-ordinates of the actual cross-profile.

Table 31 gives the co-ordinates of the cross-profile of the Brazos River in a curve computed from Formula (4) and Equation (12), together with the co-ordinates of the actual cross-profile.

TABLE 30.—BRAZOS RIVER.

Co-Ordinates, in Feet.					General remarks.
	Computed.			Actual.	
X.	*A.*	*B.*	*C.*		
255	0.00	0.00	0.00	0.00	Location 3¼ miles from the mouth.
250	0.60	0.60	0.73	0.60	
225	3.30	3.32	3.93	5.00	Area of section, 5293 sq. ft.
200	5.80	5.72	6.54	7.40	Width, 510 ft.
175	7.90	7.90	8.70	9.60	Mean depth, 10.38 ft.
150	9.80	9.82	10.47	11.60	Maximum depth, 15 ft.
125	11.40	11.50	11.91	11.60	Radius of curvature infinite.
100	12.70	12.70	13.05	12.50	
75	13.70	13.70	13.91	13.00	Value of *D* in Formula (4) = 15.
50	14.40	14.40	14.52	13.80	
25	14.90	14.85	14.88	14.50	Column *A* is computed from Formula (4), a parabola, has an area 3.9% less than the actual area.
0	15.00	15.00	15.00	15.00	
— 25	14.90	14.85	14.88	14.50	
— 50	14.40	14.40	14.52	14.00	
— 75	13.70	13.70	13.91	13.70	Column *B* is computed from Equation (11), an arc of circle, has an area 4.4% less than the actual.
—100	12.70	12.70	13.05	12.60	
—125	11.40	11.40	11.91	11.40	
—150	9.80	9.82	10.47	10.30	
—175	7.90	7.90	8.70	9.20	Column *C* is computed from Equation (12), an elliptic curve, has an area the same as the actual area.
—200	5.80	5.72	6.54	7.50	
—225	3.30	3.32	3.93	4.40	
—250	0.60	0.60	0.73	0.60	
—255	0.00	0.00	0.00	0.00	

It is an interesting fact that the inexorable laws of Nature which control the orbits of the heavenly bodies, compelling them to revolve in curves of conic sections, also control the shape of the cross-profile of flowing streams, giving it the curve of a conic section, and just as the orbit of the moon around the earth is modified by the influence of the sun and planets, so the cross-profile of channels is modified by the effect of curvature, which can be computed, and other influences in a lesser degree, but more difficult to determine and impracticable to compute.

Professor Engels' discussion‡ is important and the Society is to be congratulated in receiving a contribution from so eminent an authority. He is in general agreement with the writer's conclusions; but as to the conversion

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1911.

† *Loc cit.*, p. 1924.

‡ *Loc. cit.*, March, 1926, Papers and Discussions, p. 533.

of straight reaches into curved channels in estuaries, he regards such practice as questionable, since "the force of the tide wave would be lessened, a part of its energy wasted, and the quantity of water flowing in and out decreased, so that the scouring force of the stream would be weakened". The writer regards a suitable retardation of the tidal flow as essential to the development of its scouring force as it allows an accumulation of head so necessary to produce deepening. Although it may diminish the actual tidal flow in and out, that flow is effected by a greater water slope. In the estuary of the Delaware River the mean depth is the same in the upper reaches as in the lower ones, although the tidal flow is much less.

TABLE 31.—BRAZOS RIVER.

CO-ORDINATES, IN FEET.				
Computed.			Actual.	General remarks.
X.	*A.*	*B.*		
195	0.00	0.00	0.00	Location, 4¾ miles from mouth.
175	7.99	9.51	10.00	Area of section, 5 384 sq. ft.
150	15.58	17.31	18.00	Width, 390 ft.
125	20.68	22.31	21.00	Mean depth, 13.81 ft.
100	23.58	24.75	23.50	Maximum depth, 25 ft.
75	24.70	25.36	24.25	Radius of curvature, 2 900 ft.
50	24.26	24.55	22.50	Value of *D*, in Formula (4), 19.95.
25	22.31	23.65	20.50	Column *A* is computed from Formula (4), has an area of section 4% less than the actual section.
0	19.95	19.95	17.50	
— 25	16.93	16.71	15.50	
— 50	13.00	13.16	13.00	Column *B* is computed from Equation (12), an elliptic curve, has an area of section the same as the actual section.
— 75	9.30	9.55	11.00	
—100	5.82	6.11	9.00	
—125	2.87	3.11	6.50	
—150	0.77	1.05	4.50	This is Cross-Section No. 12 of the Official Survey of 1878.
—175	—0.23	0.26	2.30	
—195	0.00	0.00	0.00	

That the uniformity of depth in straight reaches would be disturbed by dredging operations, which Nature would proceed at once to restore, would seem to be self-evident. However, after regularization by permanent works, dredging may be desirable and even necessary to secure the required channel depth, especially where the resistance to erosion is very great. It must be understood that the uniformity of mean depth can exist only in estuaries. As far as the writer knows it is true only of the Delaware River, although he thinks it must be true of all similar estuaries.

The writer is pleased with Mr. Jordan's inquiring disposition and constructive criticism.* Doubtless more questions can be asked on this subject than the writer is able to answer; however, he may be able to clear up some of the difficulties that Mr. Jordan encounters.

To be sure the bed of a stream must be amenable to erosion else it cannot develop a shape in conformity with the forces acting upon it. When this is true, the shape of the cross-profile and the area of the cross-section are the result of the volume of flow, the velocity of the current, the curvature of the

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1141.

channel, and the resistance to erosion. It will thus be seen that the formula, by including the half width and a factor of the mean depth of the channel, does include, if not the actual velocity, the actual effect the velocity produces on any given stream. The curvature is the variable element which is provided for by its radius. The lack of uniformity in the resistance to erosion gives the cross-profile an irregularity which it would not have otherwise. A greater resistance to erosion over the entire bed of the stream causes an increased water slope and greater velocity to discharge a given volume of water through the smaller cross-section which results. Thus the factors of velocity and resistance to erosion combined give to the channel its cross-sectional area, and, therefore, the formula really contains these two factors.

The flood problem is one that still requires much study. In a sediment-bearing stream, like the Mississippi River, the channel during floods is deepened in bends and shoaled in straight reaches. When the flood subsides, the channel shoals in the bends and deepens in the straight reaches. During this transition period it is possible that the formula cannot keep pace with these changes. This condition pertains to that part of the river that transports both fine and coarse material, the former in suspension and the latter by pushing it along the bottom. In Southwest Pass, where only the fine material is transported, the channel remains practically uniform in size and shape. The material carried in suspension passes through without deposits until it approaches the mouth and the bar is formed. The material thus deposited is of quite uniform resistance to erosion and, hence, by confining the channel at the mouth to a suitable width, its shape across the bar may be computed for any given radius of curvature to a remarkable degree of exactness.

In the writer's opinion this is the most important and at the same time the most reliable use to which the formula can be put. Its value and reliability for this purpose is in no way lessened by any uncertainty about its application to other parts of the river in time of floods.

The writer is indebted to Mr. Landreth* for a comprehensive and lucid description of the actual movement of the particles of water in the bend of a stream and the manner in which those particles under the influence of velocity and curvature are able to erode the material of the banks and bed, which gives the cross-profile the shape that it assumes under these influences. However, like Mr. Jordan, he does not find in the formula any factor representing the linear velocity of the water or the slope of its surface which are the active forces that produce the results.

This subject has already been touched upon in replying to Mr. Jordan. It may be added, however, that to introduce into the formula the elements of velocity or slope would necessitate an adjustment for the size of the stream, for the degree of roughness of its bed, and for the resistance to erosion—an insurmountable task. The difficulties and complications of such a method have been well illustrated by Boussinesq.† Whereas by including in the formula only the area of the cross-section and the width of the channel the effect of all these factors is combined and there is only the relation of depth

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1139.

† "Essai sur la Théorie des Eaux Courantes."

to curvature to consider. It must be evident, therefore, that if a velocity of 0.1 ft. per sec. will not develop a channel of the same depth and shape as one of 10 ft. per sec., neither will the formula give the same depth and shape in the two cases.

The writer makes no pretension that his formulas are perfect as is evident by his effort to improve them in the earlier part of this discussion, and he welcomes any suggestions which may lead to their improvement.

In Notation 7* the writer has pointed out some of the discrepancies cited by Mr. Landreth and has attributed them to the small helicoidal action when the curvature is slight, allowing other influences to predominate. In other words, it is the actual profile and not the formula that is to blame, if it is allowable to make so discourtious a remark about Nature's performances.

Professor Haupt† has devoted so many years to the study of this problem that what he says comes from one well qualified to express a mature judgment on the subject. The writer, therefore, greatly appreciates the very favorable criticism that he makes.

Any one who has followed Professor Haupt in his extensive writings on this and allied subjects and has noted the many proposals he has made for the improvement of outer bar channels in this country and abroad, will understand his appreciation of the value of a discovery which enables an engineer to calculate quite exactly beforehand what he may expect from any given work.

It is also gratifying to get so favorable an endorsement from one as well qualified to express an opinion on the subject as Mr. Malaval.‡ As a member of the Society of Civil Engineers of France, as Ingénieur des Ponts et Chaussées, as Director of the French Company of the Port of Rio Grande do Sul (Brazil), and as Professor a l'Ecole des Travaux Publics, his standing in France is indicated; and as the author of a most excellent work, "The History of the Port and Bar Works at Rio Grande do Sul Brazil", his qualification as an analyst and his scientific instinct are clearly shown.

Captain Chisolm§ has brought into this discussion some most valuable information. Being in possession of the wonderful accumulation of data of the Mississippi River Commission, acquired by years of painstaking surveys and computations, he has been able to test the writer's formulas in a most comprehensive manner. These tests cover a stretch of the river extending from Cairo, Ill., to Fort Jackson, La., a distance of more than 1 000 miles. Although many of these tests confirm the writer's conclusions, some of them would indicate a failure to do so. These apparent disagreements will now be given consideration.

Captain Chisolm, referring to Corollary (2),* finds, at Collehe Point, La., at 81-Mile Point, and at Waverly Point, Miss., that in each case the radius of curvature is less than 40 $\sqrt{\text{area}}$ and yet these bends are not destructive.

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* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1910.
† *Loc. cit.*, August, 1926, Papers and Discussions, p. 1142.
‡ *Loc. cit.*, p. 1143.
§ *Loc. cit.*, October, 1926, Papers and Discussions, p. 1687.

A corollary is an evident conclusion from what has preceded. If the depth in a bend does not increase with an increased curvature beyond a certain amount, it is reasonable to conclude that the force which failed to increase the depth will be exerted laterally. That it does not actually produce destruction may be attributed to the fact that resistance to erosion is greater than the eroding force. That it does so in many cases is evidence that the tendency is there.

At four bends, namely, Giles Bend (Section 2371), Hard Times Bend (Section 2174), Island 16 (Section 441), and near Island No. 8 (Section 173), Captain Chisolm regards the conditions such that they disagree with the writer's conclusions as expressed in Notation 6.* Especially is this deemed to be true in reference to Section 2371 of which he says: "This does not agree with the statement that no further deepening of the channel results from increased curvature."

The conclusion in Notation 6 is too well established in the paper to be lightly rejected. It will be desirable, therefore, to examine into the conditions which exist in these bends to see whether they warrant Captain Chisolm's conclusion. It is found that these conditions are wholly exceptional in character and produce effects quite different from those found in ordinary bends. In each of these bends, except at Hard Times Bend, the conditions are complicated by an island, dividing the stream into two channels of unequal widths and depths. At Hard Times Bend, the conditions are complicated by a tributary entering on the concave side of the river just at the point of Section 2174. Any conclusion drawn in regard to this section that does not take into account the effect of this tributary stream is not reliable.

As the island complications at the three other bends are similar and as Section 2371 has the additional complication of artificial protection of its concave bank against erosion, this section will be used in the further discussion of this feature.

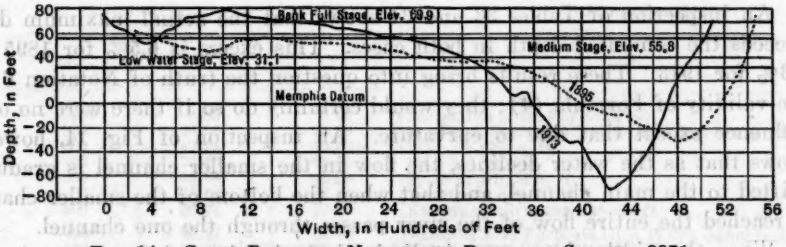


FIG. 14.—CROSS-PROFILES, MISSISSIPPI RIVER, AT STATION 2371.

Fig. 14 shows the cross-profile of Section 2371 for the years 1895 and 1913 and Table 32 gives its cross-sectional elements, as compiled by the Mississippi River Commission. These profiles, however, do not represent the actual condition at the time and stage of the river when the soundings were taken. In a river like the Mississippi, subject to erosion and accretion, the channel adjusts itself to the changed conditions as the water in the river rises and falls. This cannot be shown in a diagram embracing all stages of the water.

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1910.

The different planes of elevation and computed areas, however, are given for the different stages, namely, low water, medium, and bank-full stage, as if no change took place in the process of shifting from one stage to another. These cross-profiles, however, serve a useful purpose in showing the relative width and depth of the two channels into which the stream is divided at the two periods (1895 and 1913), and the way in which the flow in the smaller channel is transferred to the larger one as the water falls from a higher to a lower stage. Thus, the width of the two channels at bank-full stage was 5 540 ft. in 1895 and 4 290 ft. in 1913. At medium stage, these widths were, respectively, 5 080 and 2 890 ft., and at low water, 1 940 ft. in 1895, and 1 820 ft. in 1913. These variations in width plainly show that the great difference in the maximum depth on the two dates was mainly due, not to the difference in curvature, but to the great difference in width. However, to give the matter further elucidation, the co-ordinates of the cross-profile of this section for 1895 and 1913 have been computed and the curves plotted. These co-ordinates are given in Tables 33 and 34 and the curves are shown on Fig. 15 (*A*) and (*B*).

TABLE 32.—CROSS-SECTION ELEMENTS OF SECTION 2371, MISSISSIPPI RIVER, FOR THE YEARS 1895 AND 1913.

Stage.	Areas, in Square Feet.				Widths, in Feet.		Mean Depths, in Feet.		Maximum Depth, in Feet.	
	1895.		1913.		1895.	1913.	1895.	1913.	1895.	1913.
Low water..	65 650	65 650	92 100	92 100	1 940	1 820	33.8	50.6	62.0	105.2
Medium.....	80 300	145 950	54 640	146 740	5 089	2 890	38.7	50.8	....	.....
Bank-full...	75 767	221 717	149 153	195 893	5 540	4 290	40.0	45.7	....	.....

An inspection of Tables 33 and 34 shows that the actual maximum depth exceeds the computed depth in both cases. This excess is 6.5% for 1895 and 9.3% for 1913. These results bring into question the truth of Notation 6 and the validity of Formula (4); they would certainly do so if there were no other influence except that due to curvature. An inspection of Fig. 14, however, shows that as the water declines, the flow in the smaller channel is gradually shifted to the main channel, and that when the bottom of the smaller channel is reached the entire flow of the river passes through the one channel.

When the width of a stream is diminished, the channel deepens and when the volume of flow is increased without increasing the width, the same result follows. This is exactly what happens in this case. The addition of this influence to the effect of the bank protection, which prevents the normal development of the channel, gives two influences independent of the curvature, which make Captain Chisolm's conclusion in regard to Notation 6 exceedingly questionable. He must have entirely overlooked the fact of the bank protection in this bend, which was continued each year from 1897 to 1914 to prevent its movement, else he could not have classed this as a non-destructive bend.

Allusion has already been made to the fact that a cross-profile of a stream is not correctly represented when a change is made from the normal section, that is, the one in which the soundings were taken, to another stage, by the manipulation of the water surface. It will now be shown how the ratio of the maximum depth to the mean depth is affected by a reduction of the water surface to a lower plane.

TABLE 33.—MISSISSIPPI RIVER AT STATION 2371 (1913) (FIG. 15 (*A*)).

Co-ordinates, in feet.			General remarks.
Computed.		Actual.	
X.	*Y.*	*Y.*	
1 130	0.00	0.00	Computed from Formula (4).
1 000	38.56	20.00	Location, Mississippi River at Station 2371.
800	78.86	60.00	Area of section, 123 925 sq. ft.
600	100.37	96.00	Width, 2 260 ft.
400	103.79	114.00	Mean depth, 54.83 ft.
200	95.85	96.00	Maximum depth, 115.8 ft.
9	79.23	81.00	Radius of curvature, 6 660 ft. equal to $18.92\sqrt{\text{area}}$.
— 200	57.65	63.00	
— 400	34.81	43.00	Value of R in Formula (4) = 14 081 ft. equal to $40\sqrt{\text{area}}$.
— 600	15.41	27.00	Computed maximum depth, 105 ft., which is 9.3% less than the actual maximum depth.
— 800	0.18	16.00	
—1 000	—4.20	6.00	The stage of the water at the time these soundings were taken, was 47.5 ft. above Memphis Datum and 16.4 ft. above the plane indicated on Fig. 14 as low water.
—1 130	0.00	0.00	

TABLE 34.—MISSISSIPPI RIVER AT STATION 2371 (1895) (FIG. 15 (*B*)).

Co-ordinates, in feet.			General remarks.
Computed.		Actual.	
X.	*Y.*	*Y.*	
970	0.00	0.00	Computed from Formula (4).
800	28.62	22.00	Location, Section 2371, 691 miles below Cairo.
600	48.94	59.00	Area, 65 270 sq. ft.
400	57.33	60.00	Width, 1 940 ft.
200	56.39	46.00	Mean depth, 33.64 ft.
0	48.62	42.00	Maximum depth, 62 ft.
—200	36.71	37.00	Radius of curvature, 16 670 ft.
—400	23.37	34.00	Value of R in Formula (4) = 16 670, equal to $65\sqrt{\text{area}}$.
—600	11.10	20.00	Value of D in Formula (4) = 48.62.
—800	2.48	9.00	W = 970 ft.
—970	0.00	0.00	Computed maximum depth, 58 ft. which is 6.5% less than the actual maximum depth.
			The stage of the water at time these soundings were taken was 31.1 ft. above Memphis Datum: which is the plane indicated on Fig. 14 as low water.

In Fig. 1 (*A*), Brazos River, the normal ratio of the maximum depth to the mean depth is 1.445. By lowering the normal plane 5 ft., this ratio becomes 1.498. By lowering the normal plane 10 ft., the ratio becomes 1.663.

In Fig. 3 (*A*),* Mississippi River, the normal ratio is 1.426. By lowering the normal plane 10 ft., the ratio becomes 1.507 and by lowering it 20 ft., the ratio becomes 1.577.

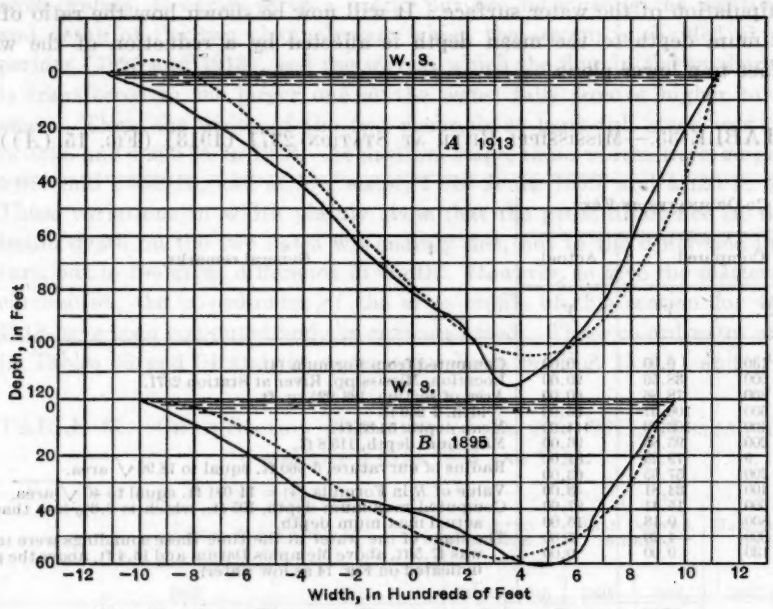


FIG. 15.—CROSS-PROFILES, MISSISSIPPI RIVER, SECTION 2371.

In Fig. 6 (*A*),†, Rio Grande de Sul, Brazil, the normal ratio is 1.425. By lowering the normal plane 4 m., the ratio becomes 1.429. By lowering it 8 m., the ratio becomes 1.528, and by lowering it 12 m., the ratio becomes 1.688.

It will thus be seen that, in every instance, the reduction of the normal water surface to a lower plane increases the ratio of the maximum depth to the mean depth, and the greater the reduction, the greater the increase in this ratio.

Now it will be noted that, in making his tests of this ratio, Captain Chisolm invariably reduces his sections to the plane of low water and therefore he obtains a greater ratio than the normal section would give. When he speaks of this range varying as much as 25% on the reach above Red River, it simply shows that a varying amount of reduction was made in this stretch of river to make his sections conform to that of low water, and in no way discredits the practically constant ratio in the normal section. It will also be noted that in his tests of this ratio in bends, he makes no allowance for the effect due to curvature, which alone causes as wide a variation in the ratio as exists in the curvatures, one from another.

It will thus be apparent that the changes in the ratio caused by the manipulation of the water surface from one plane to another will cast no doubt about the truth of the practically constant ratio which exists in all normal sections.

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1927.
† *Loc. cit.*, p. 1930.

The writer appreciates Mr. Molitor's constructive criticism* and especially that feature of it where he tries to simplify and clarify the writer's formulas. His Figs. 11 and 12† add much to their clearness. All Mr. Molitor's conclusions, however, cannot be accepted. For example, he considers that the numerical constants in the formulas must be varied to suit the differences in the character of the material through which the stream flows.

It seems to be difficult for the hydraulician to realize that the area of the cross-section of a stream is an expression of the result of the factors which produce it, namely, the volume of flow, the force of the current, and the resistance to erosion. A change in any one of these factors, while the others remain constant, will enlarge or reduce the area of the cross-section and hence the varying area expresses the varying value of these factors. This area being incorporated in the formula as has been previously explained, no variation in the numerical constants is required to provide for these changes.

Of course, a rock formation or other non-erodable material is out of the question in this discussion for no practicable current can give shape to the cross-profile of such a channel, in accordance with the law of channel development, and nothing short of artificial excavation will suffice to create the desired channel.

As regards the expedient used in the solution of Formulas (5) and (7), the writer has been unable to find any method which gives as good results as the one used. In the writer's judgment no principle in hydraulics is violated by its use.

Mr. Fawcett's discussion‡ is particularly interesting and important because he is dealing with a real live problem in process of solution. As he seems to be somewhat at sea about many features of the problem, it may be well to state briefly some of the fundamental principles that enter into it.

He has to deal with a river (the Liao River of China) of which the cross-sectional area is about 58 100 sq. ft., the width 2 700 ft., and the maximum depth about 30 ft. at low water—a river having a tidal range of 13 ft. at spring tide, a velocity of current as great as 4½ knots per hour, and a bar at its mouth extending more than 8 miles into the sea, with a depth of only 6 ft. at low water on its crest.

This river debouches at the north end of the Gulf of Liao-tung, a protected body of water extending in a southwest by south direction 300 miles, including the Gulf of Pe-Chi-Li, and having a width of 75 miles. The southwest by south winds, therefore, will produce the heaviest waves, which will strike the jetty end on, a direction most favorable for its resistance.

As stated by Mr. Fawcett an attempt is being made to deepen the channel across the bar by means of a single curved training wall, of which 35 000 ft. has already been constructed to a height of about 3 ft. above low water, and about 6 000 ft. more has to be constructed to complete its projected length. This work is progressing from the shore and seaward and there has been an advance of the bar of about 3 000 ft. in the 8 years since its commencement.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1144.
† *Loc. cit.*, p. 1146.
‡ See p. 1797.

From this fact Mr. Fawcett concludes that a single curved jetty will cause bar advance; although the writer maintains that it will not do so when given the proper order of construction and carried to a height above ordinary high water.

A paper entitled "The Economic Location of Jetties"* gives the rules for the proper construction of a single curved jetty so as to prevent bar advance. These rules require that the foundation be constructed first to the full length of the proposed jetty and then that the structure be completed to its full height, working from the outer end toward the shore. It is also shown in that paper that the single curved jetty at Aransas Pass, Texas, built in accord with these rules caused no bar advance during the 14 years of its construction, nor after its completion.

With a single curved jetty, the centrifugal force constrains the current to follow the line of the wall, and the helicoidal action causes the deepening of the channel. When the jetty is submerged, however, these forces cannot function and hence any attempt to compute the results to be expected from such a wall will be impossible, as Mr. Fawcett has found out for himself. He lays the blame to the formula, however, instead of to the submerged wall. With the jetty built to full height, however, it will create a channel across the bar of equal cross-sectional area to the channel in the river itself, and it is possible to compute its shape and size with a close approximation to the actual. This computation has been made; the results are given in Table 35 and Fig. 16. This diagram shows a channel width of 900 ft. at the 30-ft. depth at low tide.

TABLE 35.—LIAO RIVER, MANCHURIA, NORTH CHINA (FIG. 16).

Co-Ordinates, in Feet.			General remarks.
Computed.			
X.	Y.	$Y + 6$.	
1 350	0.00	6.00	Computed from Formula (5).
1 200	9.42	15.42	Location, outer bar (theoretical).
1 000	18.79	24.79	Area of cross-section of river at mean low tide, 58 100 sq. ft.
800	24.95	30.95	
600	28.26	34.26	Width of river, 2 700 ft.
400	29.21	35.21	Area of section, 6 ft. below water surface, 41 900 sq. ft.
200	28.18	34.18	Mean depth of section, 6 ft. below water surface, 15.52 ft.
0	25.61	31.61	Value of $P = 25.61$.
— 200	21.92	27.92	Radius of curvature of jetty under construction, 42 000 ft.
— 400	17.51	23.51	
— 600	12.84	18.84	Value of W in Formula (5) = 1 350 ft.
— 800	8.30	14.30	
—1 000	4.33	10.33	
—1 200	1.34	7.34	
—1 350	0.00	6.00	

The basis for this computation is the area of the cross-section of the river at a point 1 000 ft. above the inner end of the jetty. This area may not be quite exact, as the map from which it was calculated is of small scale and indi-

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 979–986 and 1103–1127.

cates only a few soundings. However, it will suffice to show the method to be pursued.

It seems hardly necessary to state that the quantities, D and P, are not interchangeable in Formulas (4)* and (5)*. One of them was devised to reproduce the cross-profile for a channel occupying the entire waterway and the other for a channel occupying only part of the waterway. To interchange these constants would make both formulas untrue.

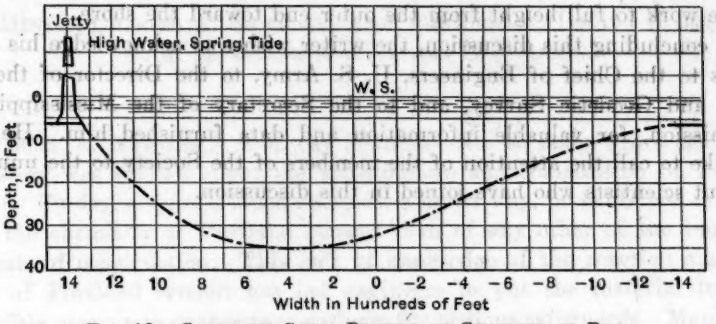


FIG. 16.—COMPUTED CROSS-PROFILE OF CHANNEL ON BAR, LIAO RIVER, CHINA.

In a curved channel the helicoidal movement causes the material excavated to be carried from the concave to the convex side of the channel. It seems evident that with only one jetty the material excavated can be more easily dispersed laterally than with a second jetty resisting the lateral flow. This will explain the fact of a single curved jetty producing a better channel than two parallel curved jetties of the same curvature.

The material excavated by a single curved jetty is not carried seaward, but is dispersed laterally causing deposits along the convex side of the channel. These deposits are continually being dissipated by wave action and sea currents.

The minimum radius to be used in Formulas (5) and (7)* must not be less than 40 times the square root of the total area of the section. (See "General Remarks" in Table 6.†)

In Notation 8‡ it is stated that:

"Formulas (5) and (7) are not applicable to straight channels nor to curved ones where the radius of curvature exceeds about 110 times the square root of the area of the channel. (The exact figure has not been determined.)"

At the time this paper was presented this was the greatest radius actually tested. Subsequent consideration of the matter, however, has convinced the writer that the maximum limit is much greater than this. Indeed he believes the limit is only reached when the centrifugal force is insufficient to constrain the current to follow the curve of the jetty or the velocity is insufficient to produce any effectual helicoidal movement.

* *Proceedings*, Am. Soc. C. E., December, 1925, Papers and Discussions, p. 1909.
† *Loc. cit.*, p. 1913.
‡ *Loc. cit.*, p. 1910.

Mr. Fawcett is to be congratulated for the excellent location given to the jetty and for the prospect that this work may prove to be a remarkable success without the necessity of a second jetty or of a resort to dredging. The increased curvature indicated for the outer portion of the jetty is objectionable. In order to secure and maintain a uniform channel, one radius of curvature should be maintained throughout. He may also be assured that any further material advance of the bar can be prevented by constructing the foundation at once to the proposed outer end of the jetty and then building up the work to full height from the outer end toward the shore.

In concluding this discussion, the writer wishes to acknowledge his indebtedness to the Chief of Engineers, U. S. Army, to the Director of the U. S. Coast and Geodetic Survey, and to the Secretary of the Mississippi River Commission, for valuable information and data furnished him. He would also like to call the attention of the members of the Society to the number of eminent scientists who have joined in this discussion.

CORROSION OF CONCRETE

Discussion*

By Messrs. Charles A. Newhall, Troy Carmichael, Dalton G. Miller, F. R. McMillan, and Nathan C. Johnson.

Charles A. Newhall,† Esq. (by letter).‡—Not so many years ago an air of mystery was thrown about the manufacture and composition of Portland cement. To-day, although much research is under way, probably less is known about the chemistry of Portland cement than of any other of the important materials of construction. This lack of knowledge of the reaction and properties of Portland cement has led engineers to put the material to many impossible uses, or to proper uses without the obvious safeguards. Many types of steel are easily rusted. No engineer would think of exposing unpainted steel of a bridge, for instance, to the elements, yet for years engineers have given no thought of protecting concrete surfaces from the numerous corrosive agencies that obviously will bring about serious weakening or even destruction of unprotected structures.

Mr. Baylis' paper comes at an opportune time when engineers and chemists all over the world are being confronted with problems arising from the failure, or threatened failure, of structures made of modern Portland cement concrete. His advice, given in the section, "What to Do",§ should be considered by all who have anything to do with Portland cement.

As far as the writer is aware, Mr. Baylis is the first to set forth clearly the part played by carbon dioxide in the hardening and aging of Portland cement concrete. It is evident that up to a certain point the formation of calcium carbonate, through the interaction of carbon dioxide and calcium hydroxide, is beneficial to concrete; for as Mr. Baylis indicates, this results in a decrease of porosity. Other reactions, not discussed by Mr. Baylis, are not beneficial and are known to take place in concrete under certain well recognized conditions.

When a concrete structure is exposed to moisture and carbon dioxide, a condition develops whereby normal calcium carbonate is first formed, followed by the change to the bicarbonate of calcium, the latter being relatively very soluble. The solubility of calcium in the form of bicarbonate is greatly increased if the water carries dissolved organic matter. All natural ground-

* Discussion on the paper by John R. Baylis, Assoc. M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

† Chemical Engr., Seattle, Wash.

‡ Received by the Secretary, August 10, 1926.

§ *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 576.

waters, sewage, and even rain water are highly charged with carbon dioxide and thus are corrosive to concrete through the bicarbonate reaction.

Calcium carbonate can occur in three forms, each differing from the other in density and in resistance to corrosive agencies, these forms being calcite, aragonite, and chalk. The first two are crystalline and the last amorphous. It is probable that "hair-cracking" and spalling of concrete surfaces are caused by volume changes in the surface layer of the concrete, resulting from the formation of calcium carbonate and the subsequent reversion of an unstable form to the stable form. It is known that after 15 to 20 years, stresses of thousands of pounds per square inch, develop in the surface layers of concrete structures when such surface is exposed to the action of carbon dioxide under conditions not yet clearly formulated.

The writer's experience of nearly twenty-five years in the manufacture and use of hydraulic cements indicates that special formula cement concrete can be made, which will not be seriously affected by the numerous corrosive agencies that are causing damage to Portland cement concrete. Such special formula cements are now being made in a small way and will be widely marketed if long-time service tests are consistent with the short accelerated tests.

The chemical formula used in the manufacture of present-day Portland cement is a development of the last twenty-five years and has been devised with the end in view of securing the maximum physical strength in the shortest possible time. No thought has been given to the resistance of the cement to the corrosive agencies to which all exposed structures are subjected. Engineers have assumed, as pointed out by Mr. Baylis, that modern Portland cement concrete was made up of compounds that are insoluble in water and like natural agencies. Benson and others* only about three years ago, were the first to point out the solubility of modern Portland cement in water. Old formula Portland cement, natural cement, pozzuolana cement, etc., all give compounds that are largely insoluble as compared with modern formula Portland cement.

Engineers long ago gave up the idea of limiting their specifications to one type of ferrous compound. Nowadays, cast iron, wrought iron, high and low carbon steels, alloy steels, etc., are specified for the particular class of service to which each is adapted. In the same manner, the writer is certain that in the future several distinct types of calcium compounds will be specified for different classes of work that call for the use of a hydraulic cement concrete.

TROY CARMICHAEL,† ASSOC. M. AM. SOC. C. E. (by letter).‡—One of the principal uses of Portland cement concrete is in street and highway construction; consequently, any paper on the corrosion of concrete has a very direct bearing on this phase of engineering.

Heretofore, it has generally been supposed—the idea having been blatantly shouted from the paid pages of the press—that the strength of Portland cement

* "Solubility of Portland Cement in Weathering Agents," by H. K. Benson, J. S. Herrick, and T. Matsumoto, *Industrial and Engineering Chemistry*, Vol. 16, 1924, p. 1063.

† City Engr., Helena, Mont.

‡ Received by the Secretary, August 20, 1926.

concrete increases as it ages. In recent years, however, an extension of the use of this material as a road surface has been accompanied by a series of failures, the failures taking place in a comparatively short time after the roads were open to traffic. Sometimes they have been attributed to causes not inherent in good concrete, the principal ones being: (1) Inadequate thickness; (2) insufficient cement; (3) improper aggregate as to quality or grading; (4) too much or too little water in the mixture; (5) the presence of organic matter in the aggregate; (6) poor workmanship; and (7) improper working conditions such as subsoil and temperature.

Wherever first-class materials are used and the pavement has been laid to adequate thickness, the ability of properly constructed Portland cement concrete to continue to maintain its original strength during successive years of road traffic has seldom been questioned by investigators into these failures, for apparently they entered into their studies with the premise that Portland cement concrete in a road, when constructed according to approved specifications and of sufficient thickness, will not deteriorate as it ages.

There are two main reasons why such a premise is erroneous:

First.—Portland cement concrete is subject to a change in its structure under impact of road traffic—the phenomenon known as "fatigue".

Second.—There is a chemical change in Portland cement concrete as it ages (which is ably described in this paper).

As to fatigue, independent of the well-known chemical change and deterioration due to the presence of sulfur or alkaline salts (of common occurrence in aggregates or seepage water in many localities) which are drawn into the concrete by capillary action, there is the phenomenon of fatigue which causes rigid masses to become weaker in structural strength under continual jarring and impact. (The well-known crystallization of steel in a car axle is akin to the breaking down of the rigid mass of Portland cement concrete in a road surface by "fatigue".)

The phenomenon of "fatigue" is touched on in an important series of tests* made in collaboration with the U. S. Bureau of Public Roads on 638 cores or test cylinders which had been drilled from apparently sound sections of California State highways, the sections from which these cores were taken totaling 800 miles. These roads had been under traffic from 1 to 7 years and the cores were tested for crushing strength. The record of tests indicated that both 1:2:4 and 1:3.5:5 concrete mixtures when laid to thicknesses of 4 to 5 in., became weaker as the pavement aged, which fact is shown by the crushing strength of the cores tested, as follows:

A.—1:2:4 Mixtures.—

1	year	old	crushing	strength,	3 800	lb.	per	in.
2	"	"	"	"	3 500	"	"	"
3	"	"	"	"	4 300	"	"	"
5	"	"	"	"	3 100	"	"	"

* Report of Study of California Highway System to California State Highway Commission, by U. S. Bureau of Public Roads, 1922.

B.—1:3.5:5 Mixtures.—

1 year old crushing strength, 3 800 lb. per in.
2 " " " " , 3 400 " " "
3 " " " " , 3 400 " " "
4 " " " " , 3 200 " " "
5 " " " " , 3 200 " " "

These recorded tests indicate that notwithstanding the fact that concrete is known to increase in strength when it is undisturbed, the actual tests of the cores of concrete taken from sound sections indicate a weakening as it ages. Except for deterioration due to alkali in the soils, or due to chemical action within the mass, this reduction in strength can only be accounted for by impact, causing the concrete to approach each year nearer the "point of fatigue" and final breaking down of the structure.

In drawing conclusions from these tests, the report makes two important deductions:

"1.— * * * average strength as determined by cores tested from concrete laid during the various years indicates that there may be a slow progressive deterioration in the concrete itself or 'fatigue' in a thin slab subject to excessive flexure.

"2.—There is * * * a question as to whether or not the concrete may be slowly deteriorating, but this condition has not been conclusively proven. More study of this phenomenon is required before any conclusion can be reached."

The author's scientific investigations are confirmed by the analysis of road conditions by disinterested road engineers. W. Calder, Chairman of Roads Board, Victoria, Australia, in his report (March, 1925) to the Minister of Public Works, Australia, on his investigation of road problems in Europe and America, makes the following pertinent statements as to the life of concrete in road construction:

"1.—While a great deal of knowledge has been gained during the past decade as to Portland cement concrete, a great deal has yet to be learned and it cannot be definitely stated that the science of concrete road building has yet passed its experimental stage or that approved standards of design are finalized.

"2.—Concrete roads designed and carefully constructed * * * are not suitable for roads subject to any considerable amount of horse-drawn traffic, under which their surface will rapidly disintegrate.

"3.—The exaggerated statements of their (Portland cement concrete pavements) durability are not justified by their service value. * * * of concrete pavements completed up to 1924 only eight (8) per cent were over ten years old and only one-quarter (¼) of one (1) per cent were fifteen years old * * *. Concrete roads are prone to temperature changes which would warrant the conclusion that their greatest utility is that of a foundation for other surface pavement.

"4.—While considerable mileage of concrete roads has been and is now being constructed in Great Britain, it cannot be said that concrete has been adopted as a standard road surface material in that country and the majority of British engineers consider that the proper function of concrete is to serve as a base for other surfacing materials.

"5.—Impact is relatively non-important, but continued impact or repetition of heavy traffic blows in great numbers will disintegrate any slab—particularly where flexure occurs. Traffic on concrete roads should be rubber-tired."

This paper opens up an important field for further research in order that the limitations of concrete may be realized and that "millions of dollars which have to be spent on repairs and replacements within the next decade" as stated by Mr. Baylis, may be minimized.

DALTON G. MILLER,* ESQ. (by letter).†—The method for the "Measurement of Mortar and of Voids in It", as suggested‡ to determine the quality of concrete for resistance to "corrosion" is interesting and the results obtained by this method, when carefully interpreted, should have considerable value. The method is, however, rather tedious of application and in this respect, when applied to freshly made concrete, has perhaps little advantage over a straight permeability test made on specially prepared specimens.

It is believed that a permeability test, properly made, has an advantage over any aggregate test of furnishing a more direct index of the actual resistance of concrete to "corrosion"; for the desired end, after all, is to produce a concrete as nearly impermeable as possible.

Regardless of how dense a concrete may be there are at least two other factors to which too little attention is given in practice, each of which may have a very far-reaching influence on the durability of concrete, regardless of quality, under special conditions of exposure. One of these is the durability of the cement used in the mix and the other, which will only be mentioned here, is the curing of the concrete.

Work in the Co-Operative Drain-Tile Laboratory at St. Paul, Minn., has shown that there is a wide difference among Portland cements in resistance to the action of the sulfates of magnesium and sodium. This difference is so great as to warrant the conclusion that certain standard Portland cements, passing all physical and chemical requirements, may be wholly unsuited for use in sulfate waters although satisfactory for ordinary purposes of concrete making. Such being the case, it may make little difference in the final results whether the concrete is well made, or otherwise, if exposed to sulfate waters, unless the particular cement used is at least of average resistance. This point is brought out in Table 14, showing results of tests of cylinders in which twelve different brands of cement were used.

The mix in all cases was 1:3 by volume and the aggregate was graded so as to have a fineness modulus of 4.67, the largest pebbles passing a ⅜-in. screen. The water-ratio was kept at 0.60 in all cases except in Series Nos. 268, 269, 271, and 273, in which it was 0.62, and Series No. 275, in which it was 0.67. All cylinders were cured for 24 hours in the moist closet followed by 20 days in distilled water, and when 21 days old were stored in the solutions, except the check cylinders, which were stored in tap-water.

The mortar of the aggregate when tested for voids by the method outlined by Mr. Baylis gave 23.9%, based on tests of a 2 by 4-in. cylinder, and 21.7% based on tests of a 4 by 8-in. cylinder, both of these results coming well below the suggested maximum of 28 per cent. These results, taken in connection

* Drainage Engr., Bureau of Public Roads, U. S. Department of Agriculture, in Charge of Co-Operative Drain Tile Laboratory, at University Farm, St. Paul, Minn.

† Received by the Secretary, September 1, 1926.

‡ *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 563.

with the results of absorption tests at 21 days and compression tests at 7 days, 28 days, and 1 year, would indicate that the concrete itself when made was satisfactory and that the difference in the results of tests of these cylinders after one year's exposure was due entirely to differences in resistance among the cements used.

TABLE 14.—STRENGTH AND ABSORPTION OF CONCRETE CYLINDERS MADE OF DIFFERENT BRANDS OF PORTLAND CEMENT AND RELATIVE RESISTANCE TO 1% SOLUTIONS OF SODIUM SULFATE (Na_2SO_4), AS INDICATED BY STRENGTH TESTS AT 1 YEAR.

(Each Result is the Average for Five, 2 by 4-Inch, Concrete Cylinders Made on Different Days.)

Series No.	Cement No.	Absorption at 21 days.	Stored in Water. Compression Tests, in Pounds per Square Inch. 7 days.	28 days.	1 year.	Stored in 1% Na_2SO_4. Compression, 1 year.	Percentage of strength ratio at 1 year.
273	62	5.62	2 480	4 470	5 760	0	0
274	33	5.74	3 170	4 880	5 950	0	0
270	60	6.07	2 300	3 850	6 380	0	0
272	63	5.75	2 390	4 610	5 950	1 900	32
269	55	6.37	2 960	4 290	5 650	3 510	62
264	39	5.84	3 280	5 020	6 080	4 790	79
268	42	6.26	2 450	3 580	6 220	4 900	79
263	37	6.26	3 180	4 820	5 740	4 600	80
262	35	5.85	3 010	4 630	6 250	5 070	81
267	40	5.60	3 310	4 600	6 480	5 310	82
275	34	6.20	3 160	4 970	5 730	5 040	88
265	41	5.51	3 460	4 790	5 860	5 930	101

No 1-year tests are recorded for the cylinders of Series Nos. 273, 274, and 270 stored in the sodium sulfate solutions, for the reason that these cylinders had completely disintegrated. Cylinders from Series No. 273 when tested at 32 weeks had 31% of normal strength.

The cylinders in this group are now (August, 1926) 19 months old and those of Series Nos. 262 to 265, 267 to 269, and 275 show little more evidence of action than they did at 1 year. At 2 years all the cylinders may show a loss of strength as compared with the 1-year tests, but at present there is every reason to believe that at least some of the series will test fairly well. When compared with 31% of normal strength at 32 weeks, as was the case with Series No. 272, in which Cement No. 62 was used, the wide difference in cements is evident.

Attention is called to the fact that none of these cylinders had any air hardening previous to exposure to these sulfate solutions. This accounts for the fact that deterioration has been so rapid. As they were all treated similarly, however, the comparative results form a fair basis of judging the relative resistance of the cements used in this work. Such being the case the necessity of considering the cement itself, as well as the grading of the aggregate for all concrete exposed to sulfate waters, becomes evident.

F. R. McMillan,* M. Am. Soc. C. E. (by letter).†—This paper is very thought-provoking on several lines and is deserving of careful study by all engineers interested in permanent concrete. The author's emphasis on the significance of porosity in the resistance of concrete to weathering is to be commended particularly.

The writer believes, however, that the value of Mr. Baylis' contribution on this latter subject will be somewhat obscured by the proposals for a new sieve size as the dividing line between the fine and coarse aggregate and for a new test for density.

The matter of the dividing size between fine and coarse aggregate does not seem to be of major importance and emphasizing it, as Mr. Baylis has done, detracts from his more important theme—porosity. The proposed test for density, offered as a solution of an important problem, can be shown to be quite unnecessary, as the information desired can be obtained in a simpler and more direct manner.

The writer believes that the author has been led to both his proposals through the conception of concrete as consisting of coarse aggregate, in which the voids are completely filled with a mortar of desired quality.

For a long time, the writer held this same conception of concrete and found it very useful in clarifying some of the problems of concrete mixtures. A real understanding of the significance of density and porosity was not had, however, until the aggregates were thought of as a single mixture thoroughly incorporated in a cement paste. Under this conception, the quality of the concrete is seen to be determined by the quality of the paste that binds the particles of aggregate together. Also, the proportions of fine and coarse aggregate and the relative amounts of aggregate and paste are at once recognized as purely questions of economy when concrete is considered in this light.

Abrams' studies leading to his law based on the relation between the water-cement ratio and strength and his subsequent investigations, as well as those by other investigators, have clearly established that the strength of concrete from a plastic mix is a function only of the water-cement ratio for given materials and conditions of test. A study of the results of Professor Withey's‡ tests of permeability shows unmistakably the preponderant influence of the water-cement ratio on the resistance of concrete from plastic mixes to the passage of water under pressure.

Fortified by these indisputable test results, the writer, for some time past, has been studying the question of density and porosity in the hope of finding a conception of the inter-relation of the various factors that would not only correlate present knowledge, but would make clear the paths for further improvement in concrete practice. The conception already given is the result of this study.

To illustrate what this conception means in the study of concrete mixtures, Figs. 15 and 16 have been prepared. The data for these figures are taken from

* Mgr., Structural Bureau, Portland Cement Assoc., Chicago, Ill.

† Received by the Secretary, September 17, 1926.

‡ "Permeability Tests on Broken Stone Concrete," by Morton O. Withey, assisted by Christopher A. Wiepking, *Bulletin 1245*, Univ. of Wisconsin, Madison, Wis.

some recent tests made at the Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill. In Fig. 15 are shown the data for a number of mixes in which the water-cement ratio was 6½ gal. per sack of cement. The scale of abscissas is of no significance, the mixes being arranged at equal intervals in the order of their densities. The ordinates to the diagram represent proportionate parts of the whole volume of concrete occupied by the different ingredients. These proportions are in terms of absolute volumes. Fig. 16 represents in a similar manner the data for two mixes, 1:1.5:3 and 1:2:4, carried through several water-cement ratios.

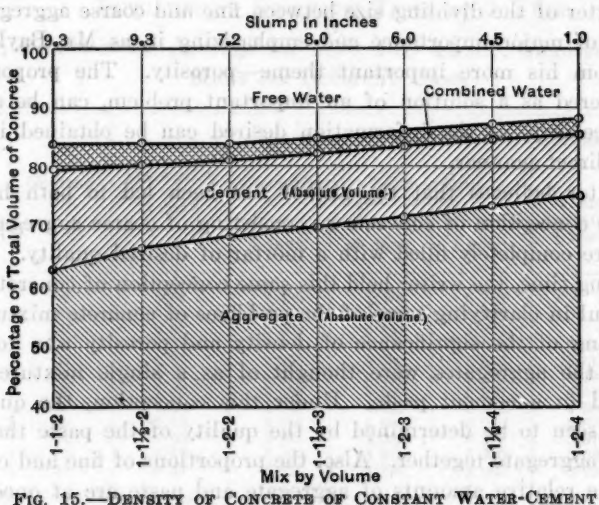


FIG. 15.—DENSITY OF CONCRETE OF CONSTANT WATER-CEMENT RATIO, 6½ GALLONS PER SACK, ALL MIXES BEING ON BASIS OF DRY COMPACT AGGREGATES.

The data used in the construction of these diagrams are all directly from the test results, except as to the quantity of combined water. This is an arbitrary quantity selected only as an illustration. The test specimens had been broken some months before this method of analyzing mixtures was conceived so that it was not possible to determine this factor. The proportion of combined water shown on Figs. 15 and 16 is taken as 0.3 of the absolute volume of the cement or about 10% by weight. It can be shown that the combined water can vary anywhere from little or nothing to two or more times the quantity indicated in these diagrams, the exact quantity depending on the length and condition of curing.

It will be apparent at once from the foregoing that the essential quality of concrete is governed by the relative amounts of cement, combined water, and free water in the paste, the part played by the aggregates being largely that of reducing the cost. In what follows brief mention will be made of the relative importance of grading or proportioning of fine and coarse aggregate.

It is not so obvious, but none the less true, as can be shown by ample test records, that for plastic mixes the sum of the absolute volumes of materials used, including the water (corrected for absorption and loss in manipulation),

exactly equals the quantity of concrete produced. In all the mixes in Figs. 15 and 16 and in many others tried in this study, when the absorption in the aggregates was taken into account the yield computed in this manner agreed within a very small margin with the yield determined in the usual manner from the weights of the test specimens at 24 hours. This small difference in every case could be accounted for by the losses in manipulation and by water settling out during the first few hours. It should be noted that, for mixes that are not plastic, when the absolute volumes are added in this manner the sum is found to be less than the original volume of the mixed aggregate, indicating that there is not sufficient paste.

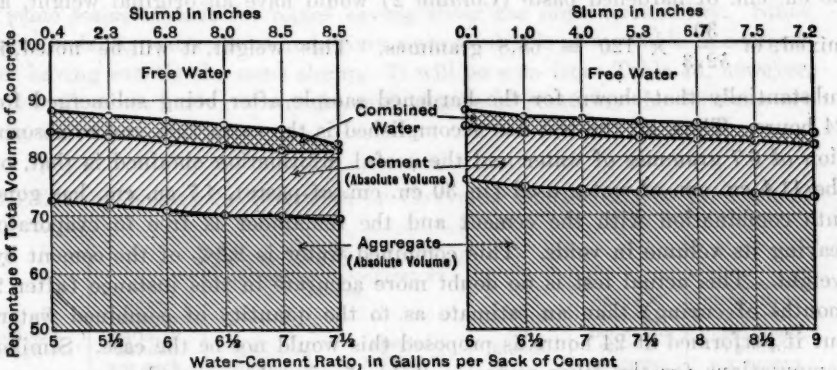


FIG. 16.—DENSITY OF CONCRETE OF VARIABLE WATER-CEMENT RATIO. MIXES ARE ON BASIS OF DRY COMPACT AGGREGATES.

The fact that the yield of concrete can be so accurately computed from a knowledge of the material entering the mixture is of importance in this discussion for two reasons: First, it materially clarifies the conception of concrete mixes, as from the very nature of this computation it is apparent that concrete consists of an aggregate mass thoroughly incorporated in a cement paste, and, further, that the quality of the cement paste is seen to be intimately associated with the quantity of water; and second, it is this fact that makes the density test proposed by Mr. Baylis an unnecessary requirement.

If one of the mixes indicated in Fig. 15 be subjected to the proposed density test the combined water at 24 hours perhaps would not exceed 5 or 6% by absolute volume of the cement. During the drying preparatory to the absorption test the remainder of the free water would be driven off and the quantity would represent the actual voids. The re-absorption of water to determine the voids, if completely carried out, would require this same quantity of water. All that the test accomplishes, therefore, is, in effect, a determination of the quantity of combined water. The quantity combined at the end of 24 hours probably does not represent the total to be expected when the structure is put into service, if the curing is at all favorable. This test, therefore, may be wholly misleading, and the simple computation of density, assuming a probable value of combined water, would be more reliable.

Unfortunately, the author did not give the quantity of water used in the example illustrating the application of his test so that a comparison cannot be made. In the data in Table 5,* however, a similar comparison can be made as follows: Take, for example, the data for the first mix given in Table 5. Assume 100 grammes of cement $\left(\text{absolute volume} = \frac{100}{3.10} = 32.3 \text{ cu. cm.}\right)$ which with its 20% of water gives an original weight of 120 grammes and an absolute volume of 52.3 cu. cm. Table 5 does not show the respective amounts of material used, but by proportion it is seen that the 30 cu. cm. of hardened paste (Column 2) would have an original weight, as mixed, of $\frac{30}{52.3} \times 120 = 68.8$ grammes. This weight, it will be noted, is substantially that shown for the hardened sample after being submerged for 24 hours. Thus, all the test has accomplished is the extraction and re-absorption of 6.7 grammes of water and the useful information obtained is that, of the 11.4 cu. cm. of water used (in 30 cu. cm. of paste), 4.7 cu. cm. has gone into combination with the cement and the remainder is free to evaporate, leaving its volume in voids. This combined water is 8.2% of the cement by weight. This actual test is no doubt more accurate in this instance (after 2 months of curing) than an estimate as to the quantity of combined water, but if performed at 24 hours as proposed this would not be the case. Similar computations for the other mixes in Table 5 give the same close agreement between the original weight of fresh paste and the weight after being submerged 24 hours. In like tests made for the writer a similar small discrepancy was found. This was largely accounted for by the loss due to a slight film of water forming at the top of the mixture.

The real significance of the analysis of concrete mixes shown by Figs. 15 and 16 goes far beyond a mere test for density. Consider the almost endless studies of concrete mixtures that have occupied the attention of many individuals for some years past. In all these studies grading of the aggregates or subjects closely related to grading have been the central theme. In Fig. 15 it is seen that for a fixed water-cement ratio the sum of the absolute volumes of cement and aggregate varies only from 0.79 to 0.85 for a wide range of mixes. This range of mixes, it should be stated, includes seven out of fifteen mixes tried for this water-cement ratio. The other eight are arbitrarily excluded either because they were too stiff to puddle (slump less than 1 in.) or because the amount of coarse aggregate was less than the fine. It might be pointed out that a diagram including all fifteen mixes is exactly similar and only extends the range of densities from 0.75 for the 1:2:1 mix at one end to 0.85 for the 1:2:5 at the other.

It is the narrow range in possible densities (sum of the absolute volumes of aggregate and cement) that has impressed all students of the problem, the desire being, of course, to increase the density and thus reduce the porosity.

* *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 562.

In his discussion Professor Williams* has pointed out what has been well known, that increased cement content contributed both greater strength and impermeability, and yet gave lower density. Professor Williams has failed to point out, however, why increased cement content increased both strength and resistance to the passage of water. The reason is that, for a given workability, the richer mixes require lower water-cement ratios. The differences in workability of the mixes in Fig. 15, indicated by the slumps at the top, should not be allowed to escape notice.

By reducing the water-cement ratio these richer mixes would give somewhat higher densities, thus improving the concrete both through the lower water-cement ratio and the increased density. Table 15 shows the data from this same series of tests for mixes having about the same workability. Since in these tests the water ratios and mixes were fixed it is not possible to compare mixes having exactly the same slump. It will be seen from Table 15, however, that for all mixes having about the same degree of plasticity the densities are nearly the same.

TABLE 15.—DENSITY OF CONCRETE OF CONSTANT SLUMP (2.5 to 4.5 INCHES)

Mix, dry compact aggregates.	Water-cement ratio, in gallons per sack.	Proportion of Total Volume of Concrete Occupied by Solids.		Slump, in inches.	Sacks of cement per cubic yard of concrete.	Compressive strength at 28 days, cured damp, in pounds per square inch.
		Absolute volume of solids in aggregate.	Absolute volume of solids in aggregate plus cement.			
1 : 1 : 2	4.5	0.66	0.82	3.0	9.0	6 100
1 : 1 : 3	5	0 72	0.85	2.7	7.3	5 400
1 : 1.5 : 3	5.5	0.72	0.84	2.3	6.6	4 780
1 : 2 : 2	6	0.68	0.81	4.6	6.8	4 320
1 : 1.5 : 4	6.5	0.75	0.85	4.5	5.5	3 870
1 : 2 : 4	7	0.75	0.84	4.0	5.1	3 520
1 : 2.5 : 3	7.5	0.70	0.80	4.0	5.3	3 100
1 : 2.5 : 4	8	0.74	0.82	4.0	4.6	2 730

In view of the foregoing considerations, it must be apparent that the hope of further improvement in concrete practice does not lie in minute considerations of aggregate grading, but rather in the selection of proper water-cement ratios, in increased curing, and in extracting in some manner the excess water, this after the concrete is in place. It needs only a glance at Figs. 15 and 16 to visualize the possibilities of increasing both the strength and density, first by taking out as much of the free water as possible while the mass is still plastic and in a condition to consolidate as the water is drawn out; and, second, to bring into combination through extended curing a large proportion of that water which cannot be extracted.

Neither of these proposed methods of improvement is visionary. Both strength and impermeability have been shown to be greatly improved by longer curing. Professor Withey's conclusions are particularly emphatic in this

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1505.

respect. Abrams* has shown the importance of curing on both the strength and absorption. The following comparison is taken from these test results. For a 1:4 concrete tested at 4 months, a water-cement ratio of 8.25 gal. per sack when cured wet for 21 days gives about the same strength as a water-cement ratio of 5.75 gal. per sack cured wet for only 3 days. Also, the absorption in a period of 24 hours (age of concrete, 1 year) was almost identical for these same two concretes, 8.25 gal. cured wet for 21 days and 5.75 gal. cured wet for 3 days. When both these concretes were cured wet 21 days the 5.75-gal. mixture was 65% stronger and absorbed 38% less than the 8.25-gal. mixture.

For an example of the extraction of water from the freshly placed concrete to increase both strength and resistance to wear or weather, there is the sand-moulded artificial stone and the absorption process of finishing cement floors. An outstanding example of concrete from which water was extracted as it was placed is the Fountain of Time at the entrance to Washington Park, Chicago, Ill. This famous piece of statuary by Lorado Taft was built of concrete by Mr. John J. Earley in 1922. It is remarkable in that the thin sections and sharp protruding members and edges are as perfect to-day as when they were built. The essential features of Mr. Earley's method† is the use of a porous back form which extracts the water from the concrete as it is being placed.

To summarize these remarks it seems that the conception of concrete as an aggregate mass thoroughly incorporated in a cement paste well explains present-day knowledge of concrete mixtures and gives a clear understanding of the paths open for future progress. By this conception it is seen that the strength and endurance of concrete as it is now made is largely determined by the water-cement ratio of the paste and the extent of the curing. The grading of the aggregates, when the concrete is mixed with a fixed water-cement ratio, is principally a question of economy, offering little opportunity for further improvement; and future progress on an important scale lies largely along the lines of increased curing and the extraction of excess water from the freshly placed concrete.

NATHAN C. JOHNSON,‡ ASSOC. M. AM. SOC. C. E.—This paper on the "corrosion" or disintegration of concrete is timely and the subject is of the utmost importance. The literature of concrete has been saturated with papers on design and reports of laboratory procedures with deductions therefrom, but there has been a dearth of papers on things as they really are in commercial structures.

It is regrettable that all corrosions or disintegrations of concrete are loosely termed "failures". The study of disintegrations in commercial structures is possibly the best and most informative of all studies of concrete as a material that could be undertaken. Yet the subject has been neglected, partly because

* "Effect of Curing Condition on the Wear and Strength of Concrete," by D. A. Abrams, M. Am. Soc. C. E., *Bulletin* 2, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill.

† See "Building the Fountain of Time," by John J. Earley, *Proceedings* Am. Concrete Inst., 1923.

‡ Cons. Engr., New York, N. Y.

of the persistent use of the undeserved term "failures" and partly because, in reflex, any dissertation on failure offends the innate philosophy of the American mind, which demands successes as a steady diet, both in its work and in its play.

Yet a sane study of disintegrations in commercial structures is of the utmost importance. Such a study brings to light a wealth of constructive comment on the art as it is and on the necessary improvement of that art if repetitions of unfortunate behaviors are to be avoided in the future. Further, such a study shows how defects may be effectively repaired in existing structures and how they may not be successfully repaired. And not least, it points the way to economies in construction that will place concrete as a structural material on hitherto unrealized heights of efficiency and economy within a few years.

"Failures" as failures are not attractive, any more than people sick in body or in mind are attractive. Failures as failures are merely unpleasant comments, bill-boarded for all to see, on the contrast between the wealth of assurances an owner receives before he builds and what he actually gets for his dollars.

"Failures" as a clinical study, however, are of an importance and interest that cannot be under-estimated. The study is difficult and it must be prolonged, else false conclusions will be drawn and a phenomenon of little importance will be given rank over a matter of real and vital significance.

To classify the different types and kinds of failures, to show their origin, to evaluate their importance, and to point out the remedy for them, would be beyond the proper scope of any discussion of this paper. But a few classifications of disintegrations and their causes may be briefly made, which will be of benefit and of help to those who are interested in concrete, either as owners, who wish to know of a surety the permanent security of their investment, or as professional men or constructors, who wish actually to be in a position to secure for an owner this stability of invested funds.

Any breadth of experience in concrete makes evident the fact that disintegrations in concrete, however severe they may be, rarely mean or threaten actual collapse or total failure of the structure. The reason for this is that present standards for design rest on assumptions of such low unit stresses that even plaster of Paris as a cementing medium would bid fair, considering the masses and sections involved, to be capable of supporting any direct load likely to be imposed.

One cannot go far in a study of disintegrations without being forced to the conclusion that the construction industry to-day abuses cement rather than uses it; and that the simple and inevitable fact that Portland cement when wetted with water, sets, hardens, and gains strength with time is the actual basis of the art, rather than the skill or ability of any person or persons or organization to bring about this reaction.

This reaction and its ultimate effect always happen, and produce, when this cement and water mixture is placed in forms, the outward appearance of concrete. The significance of this comment is not lessened by recognition that construction in concrete is complex as a business, but complex only in so far as every business is complex. In the evolving of this business the art of

managing men and the art of materials handling have been stressed as the science of construction, rather than the production of a material of definite qualities, definite properties, and definite value.

The more commercial concretes are studied, the more is the conviction forced that artistry and architectural skill, or wizardry in mathematics and design, or great executive ability as the head of an organization, do not necessarily qualify any one as an expert in concrete; and that specifications and the work executed under various specifications, so long as the commonly omitted necessities for enduring construction are unprovided for, cannot be expected to endure beyond a limited time without deterioration so long as skill in design alone is assumed as the sole necessity.

Disintegrations of concrete as found in commercial structures and the conditions that gave rise to them cannot be reproduced in the laboratory. Likewise the conditions surrounding the commercial use of cement cannot be had in the laboratory, nor can laboratory usages or dicta be reproduced in the field. Further, there is a danger that the fruits of laboratory experiment, valuable *per se*, when given without caution to the fieldman, result in their being warped for commercial ends, as in the case of the successful and profitable marketing of unfit materials under the doctrine that so long as water and cement are in a certain ratio, one to another, nothing else matters. Even the water-boy on a job knows that this is not fact; and that materials, both in kind and in quality and in quantity (as well as the manner in which they are used) make a world of difference in result, regardless of this one ratio.

The art has suffered further from a conjugate of laboratory work, that is, from the fostering of the theory that strength is all important; and that strength at 28 days is an indication of fitness and quality.

And as a further mistake, but contributed by the commercial side, the mixing machinery used universally to-day is incapable of fully utilizing the remarkable abilities inherent in Portland cement, so that, to illustrate by the one property of strength alone, a mixture of materials which might be reasonably expected to give 10 000 lb. per sq. in. of compressive strength, is considered remarkable if it renders one-third that strength in the prime of its life.

From one and all of these factors arise certain unsatisfactory behaviors in concrete. Some of the various kinds and types of such behaviors have been shown and commented on by Mr. Baylis. Others have not been mentioned and some of those omitted are of the greatest practical interest and importance.

A brief classification of the various kinds and types of commonly found disintegrations is:

(*a*) Three-branched crackings, or crazings, visible in nearly all surfaces of concrete. This is the commonest type of visible disintegration and will be commented on more fully.

(*b*) Dusting and scaling on exposed surfaces—phenomena of the hydrolysis of cement itself.

(*c*) A gradual loss of coherence in the mass due to slow reaction with elements contained in the aggregates—more common than is realized.

(*d*) Loss of coherence through lack of uniformity in the mix—a common occurrence.

(*e*) Softening, followed by loss of coherence, due to the entrance of plain water.

(*f*) Softening, followed by loss of coherence, due to alkaline waters.

(*g*) Softening, due to entrance of sea waters and accompanied by various phenomena, some simple, others complex. The total loss-and-repair bill annually on sea-water structures runs into millions.

(*h*) Disintegration by bacterial action—very rare.

(*i*) Disintegration by actual electrolysis—very rare, although many disintegrations of other origins are confused with electrolysis.

To deal with these various classes and types fully would require a fair-sized book on each. Further, frost and freezing may aggravate and accentuate as a secondary influence any disintegration arising from any one of these causes, thus giving rise to other conditions which complicate any brief discussion of this general subject.

Even aside from the disintegrations listed as to type, there are other factors which relate to cement alone and to its natural behaviors which must be known and taken into consideration if there is to be a reasonable degree of value inherent in any job. Portland cement has a right to these behaviors; and it is the obligation of the users of cement to learn what these behaviors are, and to abide by them, or else to be prepared to suffer the penalty.

For instance, Portland cement, if undiluted by sand and stone, may "kick" itself to pieces without outside influences, because of the enormous energy liberated by its reaction with water and the continuance of that liberation of energy after set has taken place, through the perpetual thirstiness of cement for water.

This is well known, yet it has been a habit for decades to make over-rich in cement all outer surfaces of concrete; and to add pure cement where it is desired to unite old and new concrete. No permanent success has ever attended these ventures, nor can there be hope of success, yet tradition dies hard; and as the procedure is reputed to be "practical", it is reputable, even if it is the very essence of the theoretical—and of the mistakenly theoretical at that.

In somewhat the same manner, cement is responsible, not through possessing improper qualities, or lack of possession of proper qualities, for that commonest of all disintegrations—the three-branched, or "map" cracking of concrete surfaces. These map cracks are commonly the first outward evidences of disintegration. There must be and is a cause for them; and, inasmuch as strength tests, or any of the usual routine tests for cement and for concrete, tell nothing as to causes or as to how to control them, it behooves engineers as a start to learn all there is to be learned about this one phenomenon and even to the devising of new tests and of apparatus for tests, if these should prove necessary for a determination of fact.

It is often stated that "map cracks" are "merely superficial". Even if this is granted, what happens in this peculiar and unruly superficial layer of any concrete when it sets? And if these cracks are merely superficial, why do deep

disintegrations follow exactly the lines of these map cracks? And since cement, when set, is strong, how much stronger must certain forces be to overcome this innate strength and to bring about fracture? And how great are these forces and what is their total significance?

These are difficult questions to answer—unless more light is shed on the behavior of cements than can be extracted or cajoled from the hundreds of thousands of strength tests, recorded and available.

To the end of obtaining answers to these questions, the instrument shown in Fig. 17 was devised. Inasmuch as it measures tensions and behaviors due to tensions, it has been named the tensometer. In its elements, the tensometer comprises a scale-beam, having a balance pan on one end; and on the other end a pair of clips, between which latter is put a sheet of thin rubber, about 5 in. square.

By putting weights into the scale pan, an initial stretch, or rigidity can be created in the rubber. An initial tension on this rubber of 3 kg. will give a satisfactory rigidity. If then, this stretched rubber be coated with some substance, such as a paste of Portland cement and water, it can be determined whether or not anything is happening in the Portland cement film which might find an outward manifestation in the behavior of concrete, as pictured in photographs by Mr. Baylis.

Suppose that this stretched rubber membrane is (in commercial work) the side of a building, or the sub-base of a concrete sidewalk, or the under-slab of a concrete floor. Just as in these members of any structure, the base is fixed, so it may likewise be assumed that, due to the weights in the pan and the consequent initial rigidity in the rubber sheet, this sensitive base is also fixed, in so far as the behavior of a thin superficial coating of cement put on the rubber is concerned.

If, now, this rubber be coated with Portland cement grout in a thin coating and permitted to set and to harden, a rather remarkable behavior manifests itself. In the first place, there is undoubtedly evaporation of water taking place. Evaporation means loss of water, and loss of water means change in volume. It soon appears in the upward movement of this scale-beam as shown by its upright indicator, that loss of water (induced by a little electric fan in the experiment) or some other cause, has caused a contraction and has lifted the weight in the scale-pan.

This lifting by contraction must, therefore, be evidence of a force greater than the force initially induced in the stretched rubber. This extra force can be measured by adding weights to the scale-pan, until the scale indicator returns to its original position.

Until these extra weights are added, however, it is observed that the cement film on the rubber is intact, although the edges of the rubber have curled inward. To many, it will be at once evident that the shape of the rubber sheet is now an exact reproduction of a "dished" cement floor, and "dishing", of course, precedes cracking, in many cases of failure in concretes.

If, now, other weights are introduced into the scale-pan, at a certain point, rather suddenly, the cement film on the rubber sheet abruptly ruptures and the pointer returns to zero position (Fig. 18).

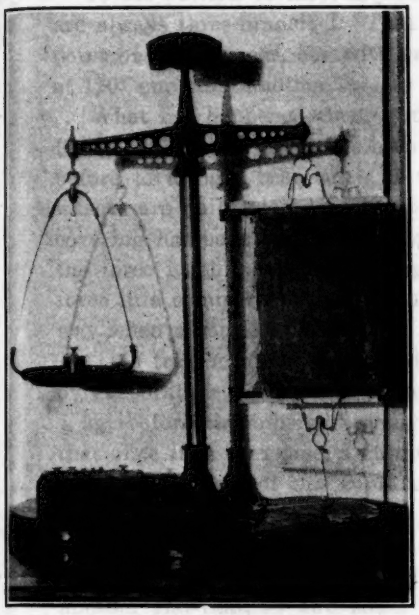


FIG. 17.—TENSOMETER, SHOWING A FILM OF PORTLAND CEMENT AND WATER ON A STRETCHED MEMBRANE, BEFORE SETTING.

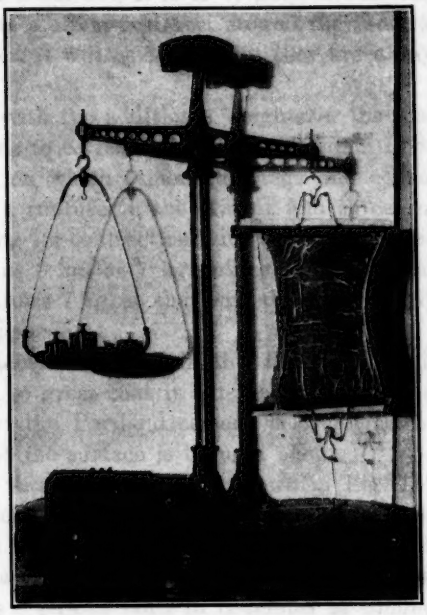


FIG. 18.—TENSOMETER, SHOWING THE FILM AFTER SETTING AND BACK-WEIGHTING TO PRODUCE CHARACTERISTIC RUPTURE LINES, BUT STILL SUPPORTING A HEAVY LOAD.

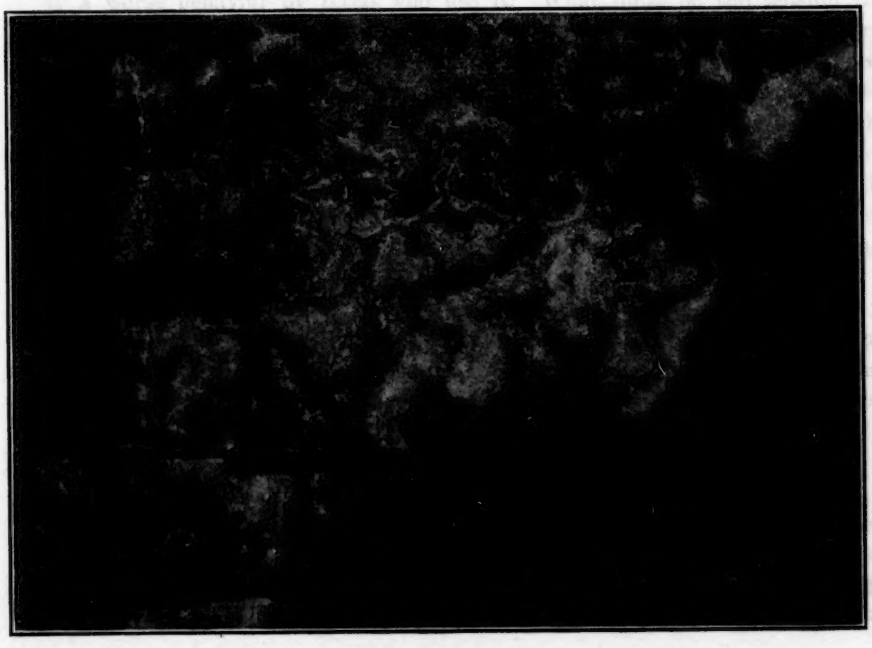
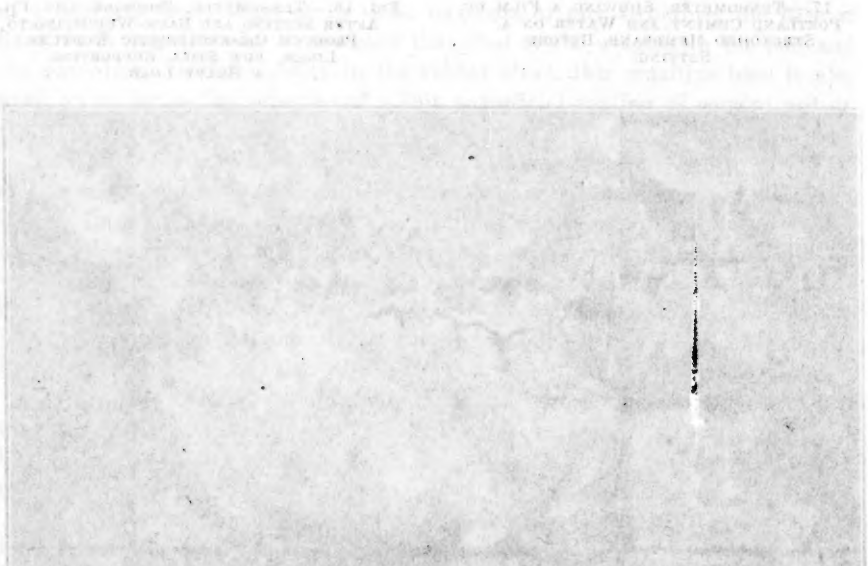


FIG. 19.—SURFACE OF CONCRETE WALL PANEL, SHOWING THREE-BRANCH CRAZING AT 120 DEGREES.



If the pattern of these ruptures is examined, it is to be noticed that they are always three-branched. That is, if a circle is drawn around the junction point of these cracks, one with another, it will be found that they are always at 120° one from another.

What has happened then, is that with the additional weights in the scale (4 kg.) the rubber sheet has been made to correspond more nearly than it did before to a rigid base, such as that on which cement toppings and cement washes are daily placed. Therefore, a rupture in the cement film or cement covering has been induced by opposing its contraction with a greater force—the force in this case being 4 kg. plus 3 kg. or 7 kg. minus. The opposing force in a commercial floor is many times 7 kg. so that something has to give way when a commercial topping is applied.

This trial means, then, that the tension induced in this thin surface film by evaporation or by other causes is so great that it lifts between 3 kg. and 7 kg. before the inherent strength of the Portland cement is overcome, but that once it is overcome, a rupture of the surface is brought about.

If the pattern of this rupture on this rubber sheet as shown in Fig. 18 is compared with the pattern in a concrete sidewalk topping or other concrete structures (Fig. 19), it could be seen that each is identical in form; and that all this "crazing" or whatever it may be called, is at 120°, one crack with another and that, by and large, it is a serious fault in concretes in commercial work.

This tension is occurring on every concrete job done in the usual manner, where cement is left to its willful and uncontrolled way, or aggravated to a worse behavior by those who use it. What must be the tension in a surface that is 50 sq. ft. in area in commercial work? Many times concrete is reinforced "against cracking," when cracking arises from a wholly different cause; and needless to add, concrete cracks regardless of this reinforcing.

Is it to be wondered, then, with this enormous tension induced in such a thin film, that rupture occurs in the usual floor or sidewalk toppings, which of course may be 1 in. or more in thickness? Or is it to be marveled that oftentimes this rupture goes deeper and actually tears apart the concrete and thus brings about other appearances shown in the paper in their final stages of an active disintegration?

By careful experimentation, it is proven that evaporation alone is not responsible for this tension, but other elements enter in; and among these elements is the reaction of the carbon dioxide of the air with the lime hydrate of the cement to form calcium carbonate. This calcium carbonate has a tension of its own entirely different from, and quite capable by itself of exceeding in value, the tension caused by evaporation alone.

This means that sprinkling to cure these evils is not, and cannot be, wholly effective, tradition to the contrary notwithstanding; but it also means that, having found out this new-old behavior natural to cement, the devising of a remedy becomes almost automatic. All these troubles are easily prevented and any concrete can now be made free of such cracks or crazings.

Similar "mysteries" about concrete may be dispelled in like manner by a little sane research and independent thought.

Evidently, then, more accurate and better knowledge of concrete is needed and the more one amasses of that knowledge and the more skilled and able one becomes in embodying that knowledge in useful, commercial constructions, the more amazing seem the every-day assertions about concrete—assertions that make up in vehemence what they lack in truth or in basis on fact.

It has been asserted that "no one cares about anything except price." That will prove a costly doctrine to those who practice it, or sanction its practice in commercial concrete work. The speaker does not believe this is true and has found, on the contrary, that those who practice the art are, in greater and growing numbers, vigorously determined that the constructions of this year and of next year and of the years to come shall prove by their unquestionable quality that the old-time, 28-day art—the trade governed by tradition and rules of thumb and uncollectable guaranties—may not and shall not stand as the best that the American nation of to-day can show to posterity.

STREAM REGULATION WITH REFERENCE TO IRRIGATION AND POWER

Discussion*

By J. D. Galloway, M. Am. Soc. C. E.

J. D. Galloway,† M. Am. Soc. C. E. (by letter).‡—The author has clearly set forth the relations between stream flow, irrigation, and power, but attention should be called to some points in the paper that need explanation. A series of diagrams is presented for different streams and localities that express the general relationship of the three factors in the form of percentages. The author, however, states§ as one of his conclusions that:

"The problem of best utilizing the available water supply for both power and irrigation is strictly a local one and cannot be treated in general terms. Definite water supplies, definite storage opportunities, definite lands to be irrigated, and known power demands are essential prerequisites for its solution."

The writer is in complete agreement with the conclusion, and for this reason believes that the data set forth in the diagrams are misleading unless properly interpreted. In all instances the elements considered are quantitative, and the percentage method does not serve to express the relation between the three. It is not correct to superimpose on the percentage curve of water supply the additional irrigation and power demand curves, if a true relationship is to be shown. The relationship between the actual quantity of water in a stream and the irrigation and power demand is lost in such a case, and the diagrams are, to this extent, incorrect. For instance, the supply of water in the river is frequently ample for all power and irrigation requirements. In most cases a large quantity of water is wasted. Any diagram, to express the relationship between the three elements, should be quantitative. For this reason, it is believed that the author would have made the subject clearer if his diagrams had expressed the relationship quantitatively and not merely by percentages.

The statement in his conclusion is correct, because each river must be studied by itself in order to obtain a true view of the entire situation. This is expressed more in detail in Fig. 14 which is reproduced from the writer's paper on "Hydro-Electric Developments on the Pacific Coast",‖ and also

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* Discussion of the paper by J. C. Stevens, M. Am. Soc. C. E., continued from August, 1926, *Proceedings*.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, May 25, 1926.

§ *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 626.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 814, Fig. 25.

from his discussion* of the paper by William Kelly, M. Am. Soc. C. E., entitled "Co-Ordination of Irrigation and Power". The diagram is made up of curves similar to those given by the author, except that a hydrograph of the stream is drawn and typical irrigation and power demand curves are superimposed thereon. The author placed considerable stress on the apparent conflict between irrigation and power in the use of the waters of a stream. It is believed that in practically all cases much of this apparent conflict disappears, or can be reconciled by the construction of reservoirs in appropriate situations.

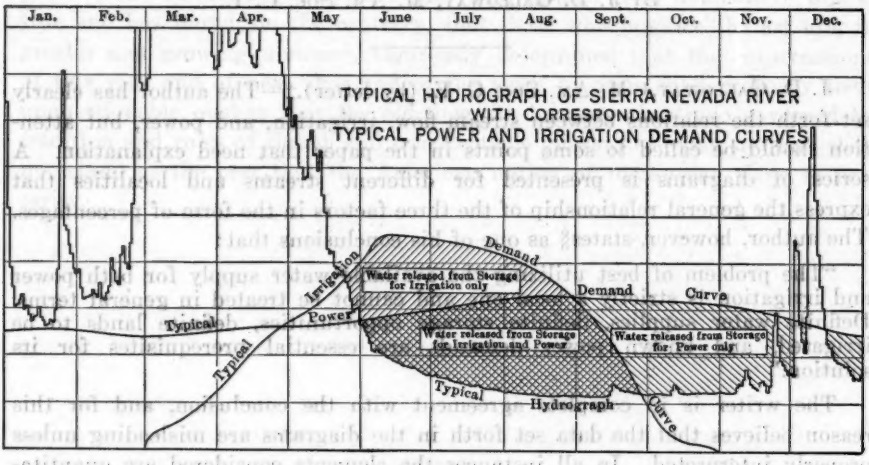

FIG. 14.

From Fig. 14, it will be noted that stored water, made necessary by low stream flow during the months, June to November, is released for two purposes, which appear in three parts of the diagram. At one time the irrigation demand exceeds the demand of power for stored water, but in September, October, and November, and frequently well into December, stored water must be released for power in excess of that demanded for irrigation. On any ordinary stream where the irrigated lands lie below the power developments, water released from storage for power purposes is available for the use of irrigation, and, to this extent, irrigation benefits by the storage of water for power. The diagram indicates that in May, June, July, and August the irrigation demand for stored water exceeds that for power, and to that extent (indicated by the area in the upper part of the diagram) stored water for irrigation must be supplied independent of the power storage. The correlation between the two, however, is indicated by that section of the diagram beneath the power demand curve, which water, released for power, is available for irrigation. To this extent, power and irrigation work together, and instead of being in conflict, the two are mutually helpful. In Fig. 14, if there were no power development on the stream, irrigation would have to provide for all the stored water indicated by the area between the hydrograph

* *Proceedings*, Am. Soc. C. E., January, 1926, Papers and Discussions, p. 147, Fig. 17.

of the stream and the irrigation demand curve. When, however, power reservoirs are installed, the burden of providing storage for irrigation is automatically lightened.

However, in addition to the correlation and harmony between the two, there is a further aid to irrigation not often appreciated. On a properly designed system, where the two interests are correlated, the irrigation reservoirs at the end of the irrigating season would be practically empty. Owing to the wide variation in stream flow from year to year, it might happen, that with irrigation reservoirs empty at the end of the season, the next winter's run-off would be so low that the reservoirs would not be filled. Ordinarily, however, where the irrigation reservoir is down stream from the power plants, the water released from the power reservoirs during September, October, and November, and sometimes in December and January, would assist in filling the irrigation reservoirs for the next year's use. In this way the stream is regulated over two or more years, and the possibility of a shortage of water for irrigation is largely dissipated by the regulation of the stream by power reservoirs.

It is admitted that the physical situation sometimes is exactly reversed, as on the Deschutes River in Oregon and in the Green River Valley in Wyoming, where all the irrigated lands lie above the best power developments. With the Deschutes River, however, although the largest part of the drainage area is tributary to the irrigable lands, the major portion of the stream flow, appearing as springs from the lava formation below the irrigable lands, is available for power without interfering with irrigation.

The purpose in calling attention to these points is to minimize what the writer believes to be the incorrect statements constantly made that the use of the waters of the stream for power and for irrigation are necessarily in conflict. The general case is that of the stream which has its source in the mountains, where power development is possible, and after delivering up its energy flows out on to a plain where the irrigable lands lie. With a proper arrangement of reservoirs—some in the high mountains for power and some in the foothills for irrigation—the regulation of the stream can be accomplished in the manner indicated and in such a way that the two are mutually helpful. The greater profits from power development will stand the higher cost of reservoirs in the mountains, and the cheaper reservoir sites in the foothills are available for irrigation. The effect is to reduce the size and cost of the irrigation reservoirs, which is practically the situation on all the California rivers, where irrigation is practiced most.

The author indicates in another way what seems to be a conflict between power and irrigation. This refers to the fact that only a small percentage of power developed on a stream is used for irrigation pumping, and the remainder (about 95%) is used otherwise. It would seem that this does not entirely explain the situation. In any country where the people depend on irrigation there is a seasonal demand for power which is made up not only of the power used for pumping, but also of that used in all the related industries that go with irrigation. One of the best examples is found in the City of Fresno,

Calif., and the surrounding region. The area is largely dependent on irrigation from several rivers coming down from the Sierra Nevada. The business of the city is to supply the wants of an agricultural community. Large packing plants are operated by power during the harvest season. The entire life of the community centers around agriculture and produces for the San Joaquin Light and Power Corporation System a summer demand greater in percentage than any other of the Western systems. In the writer's paper referred to previously, Fig. 23* shows a series of curves similar to those indicated by the author's Fig. 2.† The July and August demand of the San Joaquin Light and Power Corporation exceeds the average demand by about 27 per cent. The amount varies from year to year. All this excess demand is caused by the seasonal changes in agriculture which, in the case cited at Fresno, is almost all caused by irrigation. Power is as necessary in such a community as irrigation water.

The writer has dwelt at length on this phase of the relation between power and irrigation because it is believed that statements such as those made by the author have a tendency to emphasize a conflict that does not exist if all the factors are taken into account. Owing to the private ownership of power companies and the public ownership of irrigation supplies, demagogic politicians and short-sighted reformers have constantly endeavored to emphasize a conflict between the two, to the disadvantage of power development. It is of the same type of misinformation as that to which the author refers‡ regarding the relation between forests and stream run-off. It is apparent that water for irrigation is superior in its use to that for power. However, it is seldom the two cannot be correlated to mutual advantage, and if the total life of the community be taken into account, power is as necessary as irrigation water. It is believed to be incorrect to over-emphasize the apparent conflict between the two, and for this reason the writer has developed the same ideas which he has expressed in several discussions on the subject.

With the author's statement regarding the relation of forests and stream flow, the writer is in complete agreement. For years the public has been fed on incorrect statements of the kind he cites. Engineers cannot too often emphasize the real relationship that exists between the two.

Although some criticism has been made of parts of the author's paper, the writer desires to express his appreciation of the clear manner in which the subject has been set forth.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 810.

† *Proceedings*, Am. Soc. C. E., April, 1926, Papers and Discussions, p. 617.

‡ *Loc. cit.*, p. 624.

INCREASING THE EFFICIENCY OF PASSENGER TRANSPORTATION IN CITY STREETS

Discussion*

By T. Kennard Thomson, M. Am. Soc. C. E.

T. Kennard Thomson,† M. Am. Soc. C. E.—The speaker would like to illustrate two projects which would give enormous relief to Manhattan. The first (Fig. 9) is a belt-line railroad around the Island, with an elevated street, 100 ft. wide, above it, and an automobile speedway, of the same width (it is shown as 36 ft. wide in Fig. 9), above the elevated street. The speedway should only come down to the elevated street level, say, every 2 or 3 miles.

Fig. 9.

Obviously, the present street level is the most economical place to put the railroad tracks, which should be at least six in number. The outside tracks would be used only as unloading sidings; the next two as switch tracks to feed the sidings; and the center tracks for express trains—all for freight only.

An elevated unloading platform, 20 or 30 ft. wide, should extend all around the belt line, on the inside. This would enable "store door" deliveries, and

* Discussion on the paper by John A. Miller, Jr., Assoc. M. Am. Soc. C. E., continued from September, 1926, *Proceedings*.

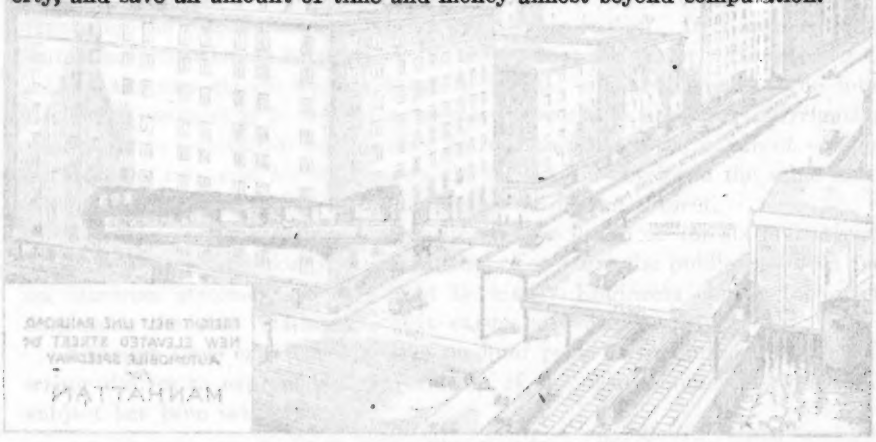
† Cons. Engr., New York, N. Y.

save from two days to two weeks, or more, of the time now required to deliver much of the freight in New York City.

The key to this problem is how to get from the elevated street to the ground. To run inclines down West Street, or down the present cross streets would ruin them, as they are not even wide enough for the present traffic. The solution would be in building inclined ramps through the center of the blocks. If at first these ramps were built every 2 or 3 miles, the property owners in every block would soon demand similar ramps in their buildings. Not only would this enormously reduce the time and cost of handling freight on Manhattan, but it would enable automobiles to go down town with speed and comfort, relieving the congestion on other streets. In winter it would have a further advantage of providing two streets unhampered by snow.

The second project is to run 42d Street under Fifth Avenue. As Fifth Avenue at 42d Street is 10 ft. higher than Madison Avenue (on the east) and Sixth Avenue (on the west) its grade would be lower than it is now. The same solution, of course, could not be used at all Fifth Avenue crossings, but it would be quite possible to treat each crossing as a separate part of the whole problem in such a way that traffic up and down Fifth Avenue would never have to stop for the cross-town vehicles, any more than the latter would be delayed by the Fifth Avenue business.

These two solutions would increase the taxable value of New York property, and save an amount of time and money almost beyond computation.



STRAIGHT LINE PLOTTING OF SKEW FREQUENCY DATA

Discussion*

By Messrs. H. Alden Foster and A. Streiff.

H. Alden Foster,† Assoc. M. Am. Soc. C. E.—Perhaps the time is not far off when some enterprising engineer or philosopher will write a book on the theory of probabilities as applied to engineering and statistical work, in such a simple and unadorned manner that it will be easily understood by all engineers. Until then, the engineer who wishes to study data that are adapted to analysis by probability methods must learn how to do so through the papers published by the professional societies.

The author has made a new step in the discussion of engineering probabilities, and one which should be useful to the profession. The method of analyzing the data and preparing the diagrams differs from any heretofore suggested, and, undoubtedly, in certain respects, is a decided improvement over previous methods. All these methods consist in the selection of some form of smooth curve to be substituted for the irregular diagram given by the observed data. In some methods the curve is plotted entirely graphically. Other methods use a particular mathematical expression or equation, the constants of which are selected in such a way that the curve when plotted will best represent the observed quantities. In any case, the ultimate use of the curve may be divided into two classes: (1) to serve as an interpolation diagram by which the irregularities of a limited group of observations may be distributed in a logical manner; or (2) to extend the range of the observations considerably, either above or below the limits of the given data. This latter class is more common in engineering work; but the methods of analysis are equally well adapted to either purpose.

The author has developed a form of empirical equation which he uses to analyze the actual data. The constants in his equation are selected by an entirely graphical process. By an ingenious use of logarithmic plotting, he is enabled to bring the original data to the form of a straight line, provided, however, that the data actually agree with the general form of his empirical equation. In any actual case, an approximate selection of the constants is made, and a straight line is located, by eye, so as to pass through these points as closely as possible.

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* This discussion (of the paper by R. D. Goodrich, M. Am. Soc. C. E., published in August, 1926, *Proceedings*, and presented at the meeting of September 8, 1926), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

The form of equation which, heretofore, has been most used for this purpose is that developed by Professor Pearson.* His equation is derived by mathematical reasoning from certain general assumptions as to the distribution of the data—these assumptions being based on the general shape of the frequency diagrams plotted from actual observations. The selection of the constants in Pearson's equations is made by a definite process, which is not influenced by the engineer's judgment except as will be mentioned. It is apparent, therefore, that Pearson's curves have a strong mathematical basis. Elderton† makes the following statement:

"Objection to Pearson's system has not been generally directed against its practical sufficiency, but has rather been founded on the contention that the theoretical basis of the system is insufficient. It is interesting to note in this connection that in a paper read before the Statistical Society this year [presumably in 1906] Professor F. Y. Edgeworth, who has himself suggested other methods, points out that Pearson's Generalized Probability Curve appears more justifiable in the matter of *a priori* justification the longer its philosophic basis is subjected to criticism."

In spite of their sound mathematical basis, the Pearson equations are difficult to use in engineering work for two reasons: (1) the duration curve cannot in general be found because the frequency curve is in a form which cannot be integrated; and (2) the constants in the equation can be obtained, in general, only by taking the "fourth moments" of the observed data. The speaker‡ was able to overcome these objections for two types of the Pearson equations. These types are satisfactory for most classes of engineering data. The constants required for determining the equations can be readily obtained from the observed data by an entirely automatic process, the only difficulty being that, when the observations are quite limited in number, the computed values of the constants are subject to a certain amount of error.

The equations presented by the author do not have quite as sound a mathematical foundation as Pearson's equations. However, in their general shape, they satisfy the form of distribution of the data very well, and should be entirely suitable for practical use. The real test of the suitability of either system of curves is whether the data fit the plotted curve.

The selection of the method of analysis would probably depend on its relative simplicity and convenience. Mr. Goodrich's method could apparently be used to advantage for certain forms of distribution which do not fall in Type I or Type III of the Pearson curves, and which were not considered in the speaker's paper. In any case, the results obtained by one method should be about as accurate as those obtained by the other, and the precision of these results will be closely dependent on the number of items in the records used.

The use of ordinary probability paper, as originally proposed by Allen Hazen,§ M. Am. Soc. C. E., has the advantage that it shows up clearly the relative skewness of any duration curve. Mr. Goodrich's method of plotting

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

† "Frequency Curves and Correlation."

‡ "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1666.

does not show this. For this reason, it might be advisable to replot the curves on the Hazen probability paper, even when they have been originally determined by the author's method.

The speaker is favorably impressed with the author's use of the frequency diagram, and especially with the analysis of this diagram as given by Dr. Kapteyn. The construction of the frequency curve, by the present method, should furnish material for a more complete analysis of the data. This subject is, of itself, rather involved, but should offer suggestions of value in certain cases. The method of analyzing population data is also instructive.

There is one definite criticism which the speaker desires to make, namely, the frequent reference to a possible negative coefficient of variation. This is obviously based on a misunderstanding, for the coefficient of variation is essentially a positive quantity, being derived from the sum of the squares of the variations from the mean, and has the same mathematical significance as the radius of gyration of the area under the frequency curve. The speaker, therefore, would question the accuracy of the two negative values for coefficient of variation given in Table 1.*

The coefficients of variation and of skew have certain advantages which, in the speaker's opinion, are not equalled by the new factors suggested by Mr. Goodrich. The coefficient of variation is a very definite measure of the "spread" of the observations, and can be readily visualized as the slope of the duration curve when plotted on the Hazen probability paper. The coefficient of skew, likewise, gives a direct measure for the skewness of the data, and can be visualized as indicating the percentage of time by which the mean differs from the median. The term, "average range", might be useful in some cases, but does not describe the distribution any better than the coefficient of variation. The term, "quartile ratio", seems to be introduced as a substitute for the coefficient of skew, but with no particular advantages over the latter. The chief advantage of the new terms is that they can be calculated readily after the duration curve is drawn.

Mr. Goodrich's method of analysis is quite ingenious, and he has done a service to the profession in making it available for general use.

A. STREIFF,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This paper amplifies and extends a series of papers on probability of rainfall and run-off and the application of skew frequency curves which reflects distinction on the Society. The writer believes that these thorough studies are unique and original, as similar investigations cannot be found in foreign engineering literature. This discussion will approach the subject-matter from a different viewpoint.

According to the ancient Greeks the supposed inability of man to peer into the future was caused by the curious Pandora; nevertheless, ever since then attempts have been made to solve that very problem, and in many instances with considerable success. Nor is any occultism involved; it is merely necessary to know the laws governing the event under consideration, and these, if

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1073.

† Cons. Engr., Jackson, Mich.

‡ Received by the Secretary, August 10, 1926.

found, will determine the future. In other words, it is merely necessary to describe accurately the subject under investigation as a function of dimensions, time, or other variables.

Sometimes such laws are known with accuracy and the forecasts, as for instance those of the astronomers, are of uncanny precision. Sometimes they are less definite, as in engineering, and still performance usually follows expectation. Again, a great many inseparable causes are at work, called chance, in which case probability methods are used and give utilizable results.

River flow and run-off have been regarded by several investigators as belonging in the latter class, and future expectations have been based on treatment according to the laws of chance. This, it is believed, should be modified. Of course, grouping ordinates according to size, and rectifying the integrated curve by varying the ordinate scale does not necessarily involve the theory of probability. That may be done with any function—the circle, the parabola, etc. A new geometry might thus be created and the properties in the "grouped ordinate" presentation might lead to some interesting correlations of the properties of figures.

If, however, from such graphical presentations conclusions are drawn as to future expectation there must be a certainty that the fundamentals of the probability theory are satisfied. For instance, a frequency curve for part of a circle might be drawn, but obviously no conclusion may be deduced from the mean value about the probable size of the next ordinate, for that is predetermined. Thus, when the author states* that, "in other words, there is an even chance that the next succeeding record will fall within these limits", he should be certain that the river does not follow definite, separable laws, but is governed by chance. This, to be sure, has been the accepted view thus far. The assumption is indeed tempting. Nothing seems more variable than the factors which create river flow—precipitation, radiation, cloudiness, wind, barometric pressure, evaporation, transpiration, seepage, geologic formation, temperature, topography. How can the existence of separable laws be expected from such a multitude of variables? This, perhaps, explains why the subject has heretofore never been investigated.

On the other hand, indications that run-off does follow definite laws, may also be found. Professors A. E. Douglass,† of the University of Arizona, and Ellsworth Huntington,‡ of Yale University, have measured the variation in width of the growth rings of Sequoia trees and have found an oscillating variation around a mean value and a strong correlation with rainfall in some localities. Conclusions regarding the California climate for 3 000 years have been drawn. They also find correlation with solar activity, as expressed in the so-called Wolf numbers of sun spot frequency. In some instances, as in pine trees of the Eberswalde Forest, Germany, the correlation with Wolf numbers is exceedingly strong.

The remarkable fact involved is that such strong correlation with the Wolf numbers cannot be traced in the weather elements which influence tree growth,

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1077.

† "Climatic Cycles and Tree Growth," Carnegie Inst., Washington, D. C.

‡ "The Climatic Factor," Carnegie Inst., Washington, D. C.

and the causes are as yet quite obscure. Professor Huntington believes that the tree acts as an integrator of the weather elements and adds the various influences not traceable separately. If, however, the limited surface of a tree acts as an integrator of the weather elements, how much more can this be expected from a river the drainage area of which covers many thousands of square miles?

Any one who has observed the path of rainstorms realizes their fortuitous character. The drainage area, however, collects them all and will no doubt show continuous relations, if these exist. A casual inspection of run-off data dispels comparison with unrelated numbers. Hoyt* has shown that such data strongly converge around a constant mean. This also can easily be found in rainfall (see Table 17).

TABLE 17.—PERCENTAGES OF ANNUAL RAINFALL AT LUND, SWEDEN.

Average of:	Percentage of mean.	Average of:	Percentage of mean.
10 years	101.8	90 years	102.2
20 "	101.2	100 "	101.5
30 "	102.2	110 "	100.7
40 "	103.4	120 "	100.8
50 "	104.2	130 "	100.7
60 "	104.6	140 "	99.9
70 "	103.6	150 "	100.4
80 "	103.4	157 "	100

Then the variation with the seasons also strongly suggests a controlled sequence. Likewise, every hydraulic engineer is familiar with the undulating appearance of a mass curve showing yearly run-off, indicating a periodic variation and grouping in series of wet and dry years. This latter quality is the cause of the skew frequency curve. The skew frequency curve is easy to imitate artificially, by assuming a definite function somewhat resembling a run-off curve. These run-off curves are periodic curves without definite period, that is, the values oscillate repeatedly around a mean value, but never repeat themselves. This is the class of curves analyzed by so-called periodogram analysis, which differs from harmonic analysis in that the latter is applied to curves which do have a definite period. A curve resembling the run-off curve can be obtained by taking a Fourier series. The word "resembling" should be emphasized, for the run-off curve cannot be analyzed by either harmonic or periodogram analysis, but requires special methods.

For the Fourier series:

$$y = c_0 + c_1 \sin x + c_2 \sin \left(2x - \frac{\pi}{2}\right)$$

in which, x and y are the co-ordinates, and c_0, c_1, and c_2, are constants. It may be found that this is a simple form of undulating periodic curve which has one greater and one smaller maximum per period, similar to a run-off curve with spring and fall maxima. This has a skew frequency curve precisely like

* "Run-Off," by Gardner S. Williams, M. Am. Soc. C. E., in "Civil Engineer's Pocket Book."

those shown by the author. By adding a few terms or varying the constants the entire range of skew frequency curves can be obtained, which proves that these curves can be obtained under conditions of controlled sequence.

Furthermore, the probability theory furnishes several criteria proving that run-off figures are not unrelated numbers. The "Goutereau ratio" shows this. Goutereau* aided by Maillet has shown that the ratio: Mean variation divided by mean deviation equals 1.41 for Gaussian numbers. Flow data taken at random do not substantiate this ratio. The 53-year record of the Murray River, Australia, given by L. Standish Hall, Assoc. M. Am. Soc. C. E.,† furnishes a ratio of only 1.016. Marvin has recently emphasized the value of this index.‡

There is another criterion. If an event has occurred a times and missed b times in n cases, the probability *a posteriori*, $\frac{a}{n}$, is also the adopted value for the probability *a priori*. A deviation, e_1, from $\frac{a}{n}$ is to be expected with a probability, $f(g)$, according to the relations,

$$g = e_1 \sqrt{\frac{n^3}{2\,a\,b}}$$

and

$$f(g) = \frac{2}{\sqrt{\pi}} \int_0^g e^{-t^2}\, d\,t$$

This is the theorem of Bayes. Taking, for instance, $n = 365$ and $a = 104$, $e_1 = \frac{1}{20}$. Then, $g = 1.5$ and $f(g) = 0.966$; or the chances are 28.4 to 1 that $\frac{a}{n}$ equals $\frac{104}{365}$ for the next 365 unrelated numbers, within 5 per cent. This obviously is not true for rivers, for it all depends on a wet or dry year whether, for instance, the daily gauge reading will be lower than a certain height. Actual count on the Tennessee for gauge heights below the reading, 1, gave the data of Table 18. Thus, in 29 trials the count gives 2 "yes" against 27 "no", whereas the theorem of Bayes gives 28 "yes" against 1 "no". If the theorem of Bayes is true, the river flow cannot be a fortuitous sequence.

For twenty-five years the writer has cherished the idea of deducing definite laws of river flow, and for the past ten years has devoted a great deal of work to this problem. The question appeared not to be readily solvable, for rivers would yield no results if treated with the existing means of periodogram analysis. It became necessary to devise special methods in order to obtain results, until finally the laws of river flow were found.

The subject is too voluminous to be presented fully in this discussion Briefly, the main laws of river flow are as follows:

* "Sur la variabilité de la température," *Annuaire* de la Société Météorologique de France, 1906.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 246, Table 16.

‡ *Monthly Weather Review*, September, 1924.

1.—Hydrographs can be resolved into a definite number of elementary curves, which are either homogeneous or damped oscillations.

2.—The same elements recur in different rivers, but the phase and amplitude change.

3.—These elements may be divided in two groups. There are no others.

4.—The first and most important as to size is due to the earth's motions. This group does not change the mean yearly run-off from year to year.

5.—The second group causes the variation from year to year. This group is identical with elements in the curve of the Wolf numbers of sun-spot frequency.

6.—Run-off follows the long climatic cycle described by Brückner,* the so-called Brückner cycle, which is identical with the second largest element in the curve of the Wolf numbers. The low levels in the Great Lakes at present are produced by the same cycle.

A paper on these laws has been prepared by the writer and published.†

TABLE 18.—Frequency of Daily Gauge Readings Below 1 Foot, Tennessee River at Florence, Ala.

Year.	Frequency.	Year.	Frequency.	Year.	Frequency.
1881	104	1890	0	1900	97
1882	22	1891	0	1901	30
1883	37	1892	0	1902	101
1884	33	1893	9	1903	78
1885	43	1894	4	1904	147
1886	6	1895	84	1905	88
1887	69	1896	123	1906	42
1888	95	1897	73	1907	5
1889	0	1898	90	1908	41
		1899	68		

In view of these laws the sequence of run-off years cannot be regarded as fortuitous. For example, in Southern Michigan increased run-off is expected for the next two years, in accordance with the strong correlation with the Wolf numbers. This conclusion of course involves the use of probability, which is, therefore, not to be dispensed with, but merely modified in its application by laws of river flow. Nor is the clever presentation of the author thereby impaired. It is possible that the relation between extreme values remains the same in the original data and its "grouped" presentation, regardless of the fact that the data are values of a definite function. This point will no doubt be clarified in due time.

It is impossible to draw conclusions as to the expectancy of next year's run-off; but this point is of little consequence as far as the application of the frequency curve is concerned, for its chief use lies in the estimate of maximum and minimum conditions. These are obtained directly and neatly by resolution into cycles. That phase combination which produces a maximum or minimum can easily be determined by mere inspection, and this gives the desired quan-

* "Klimaschwankungen seit 1700," Vienna, Austria, 1890.

† "On the Investigation of Cycles and the Relation of the Brückner and Solar Cycle," *Monthly Weather Review*, July 1926.

tity by means of a short record of twelve years. A correction for the longer cycles, namely, the Brückner and the longest so-called secular cycle (which are, compared with the shorter cycles, very flat), can be estimated. Their influence is not material from year to year, and is of real importance only in cumulative effects, such as lake levels.

It would appear at first improbable that extreme values can be determined from a record of only twelve years, but it is easily explained as follows: The elements of river flow are incommensurable, as, for instance, $\sin x$ and $\sin x\sqrt{2}$ and, therefore, a long time may elapse until the elements are combined in nearly the hand-picked combination which produces the maximum or minimum. This constitutes the great value of the writer's method over the frequency curve, for it is hardly possible with the latter to obtain a clue regarding maxima or minima with a twelve-year record. A resolution into cycles will accomplish just that.

The foregoing does not apply to flood flows—this has not been investigated, at least not by the writer. For the present it applies only to monthly and yearly averages. Eventually the flood flows too will perhaps yield to such treatment and expose the laws under which they occur.

DISTRIBUTION OF REINFORCING STEEL IN CONCRETE BEAMS AND SLABS

Discussion*

By Messrs. James B. Newman, E. F. Rockwood, C. A. Willson, F. Theodore Mavis, and Jacob Feld.

James B. Newman,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper reveals much of interest and value, although the writer differs as to some points. He feels that the moments at the center and the supports are more the uncertain factors than the distribution of steel which necessarily follows, and that this problem with its limitations has been quite fully treated in current literature and texts. The following brief criticisms, therefore, will not be fully developed, references being furnished to more complete discussions.

The author has used many computations in showing for several numerical cases that the sum of the center moment and the average of the negative end moments of a beam carrying a uniformly distributed load of W per lin. ft. is $\frac{W L^2}{8}$. It is clear, by referring to Fig. 9 (*a*), that a beam, $A B$, carrying a uniformly distributed load of W per ft., and terminal negative moments, which may or may not be equal, is exposed to positive and negative moments as shown throughout the span. The resultant moment at any point, the algebraic sum of the positive and negative moments, is shown with respect to the base line, OO, Fig. 9 (*b*). It is clear from Fig. 9 (*b*), as pointed out in most texts, that the sum of the center positive ordinate and the average of the end terminal moments is $\frac{W L^2}{8}$. It should be emphasized, however, that this diagram only provides for a single position of the load, whereas even in building work a considerable portion of the live load may be moving, and while the complicated criteria for maximum shears and moments which hold in bridgework are out of place here, some reasonable consideration may be given to varying positions of the live load.

Theoretical considerations of continuous beams resting on simple supports§ and of continuous beams in monolithic constructions‖ show that, for vary-

* Discussion of the paper by Boyd S. Myers, M. Am. Soc. C. E., continued from October, 1926, *Proceedings*.

† Asst. Prof. of Architecture, Univ. of Michigan, Ann Arbor, Mich.

‡ Received by the Secretary, August 31, 1926.

§ "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, and other texts.

‖ Concrete Engineer's Handbook, by Hool and Johnson.

ing ratios of live and dead load and varying positions of the live load in successive or intermittent spans, the positive or the negative moment may cover practically the entire span. Moreover, the maximum value that the negative moment may readily attain, plus the maximum value that the positive moment may attain, considerably exceeds $\frac{WL^2}{8}$. In fact, very reasonable values which may be attained as to magnitude and distribution are shown

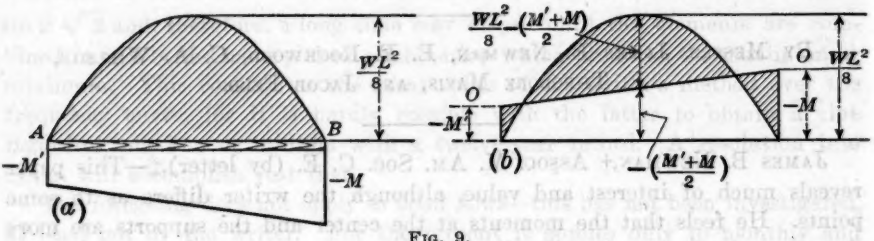

FIG. 9.

in Fig. 10. It is true of course that such positive and negative moments could not exist simultaneously, but as either the positive or negative moments here shown can be attained singly, the design should be suitable to provide for both. In this case, therefore, the total steel required would be based on $\frac{WL^2}{6}$.

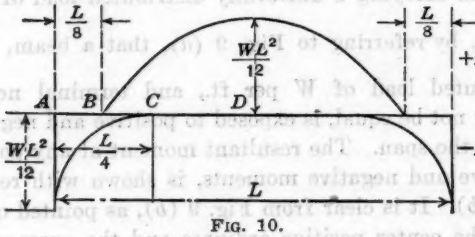

FIG. 10.

The author assumes that the moment transmitted to the column is a direct function of the negative steel in the beam at the support instead of being simply limited by that steel. The monolithic structure is statically indeterminate to be sure, and as a practical expedient, a certain moment may arbitrarily be assigned to the beam at the ends, obtaining the center positive moment by subtracting the assumed end moment from a reasonably determined total moment, and then designing the structure to adequately care for these moments. The design will then be consistent, and safe, but this does not mean that it will be either rational or good. Some parts may be seriously over-stressed, as the author pointed out in the case of the monolithic beam with all positive steel.

A steel bent carrying a single transverse load, for example, may be so designed that the member, *DE*, Fig. 11 (*a*), carries the entire load. However, distortion of this member will throw stress into the member, *FG*. The

bent may be designed as in Fig. 11 (*b*), with *EF* and *FG* carrying the load, but then under distortion, *DE* will be over-stressed. It may be assumed that *DE* and *FG* carry the stress equally, Fig. 11 (*c*), although due to shortening of *EF* it is not true. While all these structures will stand if consistently designed for these conditions, all are not equally rational. The actual determination of the moments at the supports and the center of the beam is a matter of mechanics and not of assumptions. It is only because some of the practical factors which enter into the situation are unknown, and must be appraised that there is any justification for arbitrary assignment, but assumptions should approximate the true values, as nearly as they can be reasonably worked out, from consideration of statical laws.

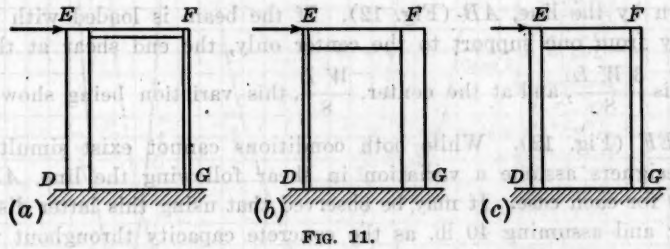

FIG. 11.

The moment the beam carries at the support must be adequately provided for, however troublesome that may be. As the author suggests, some bottom steel may be used, although this is expensive, and the section may be designed as a doubly reinforced beam; additional depth may be gained by haunching; or as is usually the case, this section may be designed as a rectangular beam, thus determining the width and depth of the stem throughout. This may give a larger concrete section than is actually required at the center, based on the T-section. The stresses at the support are often taken a little higher than at the center, on the theory that the maximum stress falls off much more rapidly than in the center where the moment curve is relatively much flatter; also because of the column effect, both in providing a relatively wide support, and, by construction, tending to reduce moments theoretically deduced. It should be mentioned, however, that this column effect has already been partly discounted in the assumption of moment factors shown in Fig. 10.

The author's discussion of shearing stresses and stirrups is along the generally accepted lines and requires little comment. There is ground for some difference of opinion here too, however. The stress that the stirrups are assumed to carry is the vertical component of the tensile principal stress. The longitudinal or transverse shear as ordinarily computed is used as a gauge of this diagonal tensile stress and, therefore, it is said in common parlance that the stirrups carry the shear not carried by the concrete. The concrete beam as noted is non-homogeneous, and the actual principal stresses set up where there is considerable shearing stress and where the steel is assumed to carry all the tensile stress due to bending (although the concrete carries some, depending on the amount it has been cracked) cannot be known with any great degree of accuracy. It is known, however, that if

the stirrup steel inserted is ample, the beam distortion is kept to a minimum and the beam acts more as if homogeneous. This shear reinforcement, for the good it does, is relatively inexpensive, and there is accordingly some justification for not skimping material. This is one justification for using the concrete to carry only one-third the shearing stress, to the point at which the shear is 40 lb. per sq. in., instead of a straight 40 lb. throughout.

Moreover, as in the case of the moments already considered, the author assumes the entire load to be static. A reasonable variation in shear may be briefly noted, however. A simple beam carrying a uniform load, W, per ft., carries a total end shear of $\frac{W}{2}$, and a center shear of O, the variation being as shown by the line, AB (Fig. 12). If the beam is loaded with the same intensity from one support to the center only, the end shear at the loaded support is $\frac{3}{8}\frac{W\,L}{}$, and at the center, $\frac{W\,L}{8}$, this variation being shown by the line, DEF (Fig. 12). While both conditions cannot exist simultaneously, some designers assume a variation in shear following the line, AE, which provides for each case. It may be observed that using this latter distribution of shear and assuming 40 lb. as the concrete capacity throughout will tend to approach a shear variation from A to B, with the concrete carrying only one-third the stress to a point at which the unit shear is 40 lb.

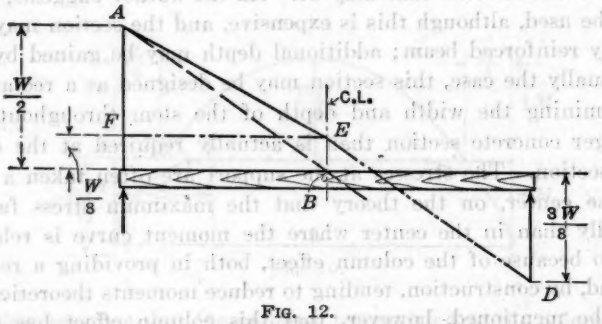

FIG. 12.

The convenient method of determining stirrup spacing used by the author has also been suggested by Turneaure and Maurer in their text, and a graphical division of the shear triangle into the several parts has been suggested by Hool and others.

It should be pointed out that any methods of determining stirrup spacing are not so easily applied nor so simply carried out when the shear curves show discontinuous and irregular variation.

E. F. ROCKWOOD,* M. AM. SOC. C. E. (by letter).†—The author is not presenting anything new, as his recommendations were discussed and discarded by leading engineers twenty years ago. Herewith is a plot (Fig. 13), pre-

* Boston, Mass.

† Received by the Secretary, September 16, 1926.

pared by the writer at that time, to show one very serious error in Mr. Boyd's reasoning.

It was known and admitted then that the moment at one end of a fixed beam plus the moment at the center equals the moment at the center of a simple supported beam of the same span and load conditions. Unfortunately, however, the distribution of moments between these two points is determined by the flexure of the beam and not by the whim of the designer.

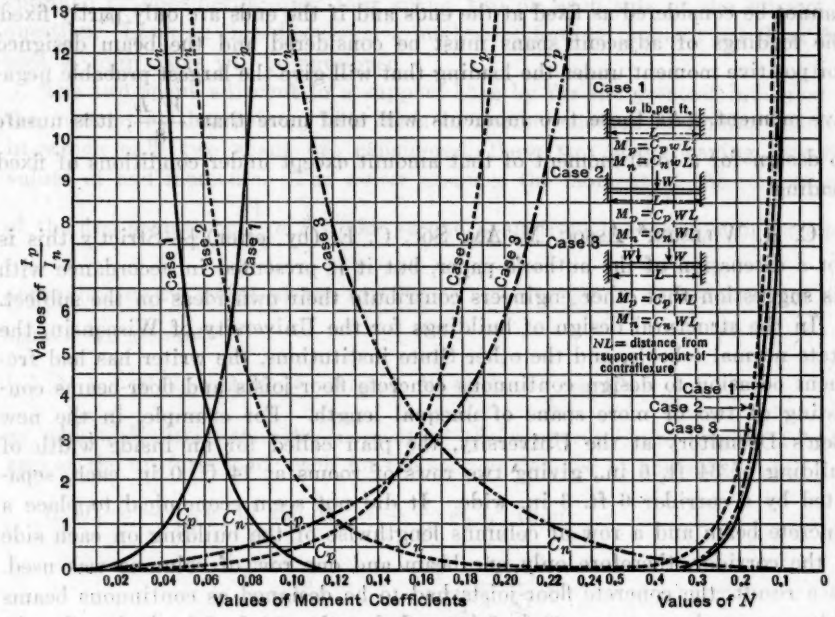

FIG. 13.

If the beam is of uniform section throughout, the end moment is $\frac{W\,L}{12}$ and the center moment is $\frac{W\,L}{24}$. Mr. Boyd recommends $\frac{W\,L}{12}$ at the center and $\frac{W\,L}{24}$ at the supports. Therefore, it may be instructive to assume a beam designed in accordance with his recommendations and check it.

Let the ends of the beam have a section of 10 by 20 in., with 1% of steel. Stresses of 16 000 and 750 lb. per sq. in. are allowable. If $n = 15$, the negative moment of inertia, I_n, is $0.075bd^3 = 6\,000$. At the center assume the same stem and an 8-in. slab. The corresponding flange width is 138 in. and the percentage of steel is $\frac{4\,100}{138 \times 20} = 0.145$. The positive moment of inertia then becomes $0.01 \times 138 \times 20^3 = 11\,040$. Referring to Fig. 13, for a ratio of $\frac{I_p}{I_n} = \frac{11\,040}{6\,000}$, or 1.84, M_n becomes $0.05\ W\,L$ and M_p becomes $0.075\ W\,L$.

Mr. Boyd provides negative reinforcement corresponding to only 0.042 WL, and if the slab is thinner, I_p and M_p will decrease and M_n will increase until it becomes 0.083 WL when $I_p = I_n$.

Many engineers would like to make the total steel in a concrete beam equal to $\frac{WL}{8}$ although they might, for the reasons herein stated, use a distribution different from that of Mr. Boyd. However, except for beams of relatively small section compared with the columns into which they frame, most beams cannot be considered as fixed at the ends and if the ends are only partly fixed, the loadings of adjacent spans must be considered and the beam designed for positive moment under the loading that will give the largest probable negative moment. As these two moments will total more than $\frac{WL}{8}$, it is unsafe to design for a total moment of that amount except under conditions of fixed loading.

C. A. WILLSON,* ASSOC. M. AM. SOC. C. E. (by letter).†—Strictly this is not a discussion of the author's paper, but it is presented in accordance with his suggestion that other engineers contribute their own ideas on the subject.

In the structural design of buildings for the University of Wisconsin, the State normal schools, and the other State institutions, the writer has had frequent occasion to design continuous concrete floor-joists and floor-beams consisting of two or more spans of unequal length. For example, in the new Men's Dormitory at the University, the plan called for an inside width of building of 34 ft. 6 in., giving two rows of rooms at 14 ft. 0 in. each, separated by a corridor 6 ft. 6 in. wide. It did not seem economical to place a concrete beam and a row of columns lengthwise of the building on each side of the corridor, therefore only one beam and one row of columns was used. As a result, the concrete floor-joists had to be designed as continuous beams of two unequal spans, one 20 ft. 6 in. and the other 14 ft. 0 in. in length. At the new Nurses' Dormitory at the University the window treatment required an unequal spacing of wall columns. This meant that the spandrel beams had to be designed as continuous beams consisting of a series of unequal spans.

Moments at Supports.—Cases like this have been so numerous that the writer has found it necessary to develop methods for determining the moments in continuous beams of unequal spans with the maximum of speed and a relatively high degree of accuracy. In the design of continuous beams of unequal spans, the major problem is to determine the moments at the supports—the remainder of the design is comparatively simple.

Quite often in building construction, the maximum load per linear foot on a continuous joist or beam is the same for all spans. Theoretically, for a maximum moment at any support the two adjacent spans and each alternate span should be loaded.‡

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† Received by the Secretary, September 13, 1926.

‡ See diagrams in "Modern Framed Structures," Vol. II, "Statically Indeterminate Structures and Secondary Stresses," by Johnson, Bryan, and Turneaure, p. 35, or in "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, p. 299. These diagrams show influence lines for moments at interior supports of continuous beams.

However, the load in the second span from any particular column has only a small effect on the moment at that column and any ordinary load on the third span will have practically no effect. Consequently, the writer uses a constant value for the load per foot for all spans in ordinary problems because the work is simplified thereby with only a slight sacrifice in accuracy.

Influence lines show that an excellent notion of the negative moment at any support of a continuous beam composed of a long series of spans may be obtained by considering only four spans, two on each side of the support in question. Later in this discussion a method of finding moment based on this idea of four spans will be developed.

The two spans adjacent to a support have by far the greatest influence on the moment at that point. This leads to another method of finding moment in which only two spans are considered, these two spans having assumed values of end moments. The writer assumes the moment at the remote end of the first span to be $\frac{1}{12} w l^2$ if the second span from the column in question approximates the length of the first span. A value of $\frac{1}{24} w l^2$ is used for end spans. In general,

$$M_2 l_2 + 2 M_3 (l_2 + l_3) + M_4 l_3 = -\frac{1}{4} w_2 l_2^3 - \frac{1}{4} w_3 l_3^3$$

in which, M designates the moment at the support; l, the length of span; and w, the uniform load; each for the span as denoted by the subscripts.

If $M_2 = -\frac{1}{12} w_2 l_2^2$ and $M_4 = -\frac{1}{12} w_3 l_3^2$, then,

$$M_3 = -\frac{w_2 l_2^3 + w_3 l_3^3}{12 (l_2 + l_3)} \quad \ldots\ldots (8)$$

However, if $M_2 = -\frac{1}{24} w_2 l_2^2$ and $M_4 = -\frac{1}{24} w_3 l_3^2$, then,

$$M_3 = -\frac{5}{48} \frac{w_2 l_2^3 + w_3 l_3^3}{l_2 + l_3} \quad \ldots\ldots (9)$$

Let $w_2 = w_3$, $l = l_2$, the long span, and $n l = l_3$ the short span. If Equations (8) and (9) are to be expressed in inch-pounds instead of foot-pounds, then,

$$M_3 = -w l^2 \left(\frac{1 + n^3}{1 + n}\right) \quad \ldots\ldots (8a)$$

$$M_3 = -1.25\, w l^2 \left(\frac{1 + n^3}{1 + n}\right) \quad \ldots\ldots (9a)$$

Now, let $C = \frac{1 + n^3}{1 + n}$. Then, again, in inch-pounds,

$$M_3 = -C w l^2 \quad \ldots\ldots (8b)$$

$$M_3 = -1.25\, C w l^2 \quad \ldots\ldots (9b)$$

The relations between the long span, short span, and $C l^2$ are shown in Fig. 14. It will be noted that as far as this diagram is concerned the short

span is never less than one-half the long span. The value of C is a maximum of 1.00 when $n = 0.00$ and 1.00; it is a minimum of 0.75 when $n = 0.5$; and it is symmetrical about the point at which $n = 0.5$. For instance, $C = 0.84$ when $n = 0.2$ or 0.8; and $C = 0.76$ when $n = 0.4$ or 0.6. Consequently, when the short span is less than one-half the long span, Fig. 14 may still be used by subtracting the short span from the long span to get an equivalent short span producing the same moment at the interior support. With a long span of 22 ft. and a short span of 8 ft., determine the value of $C\,l^2$ from Fig. 14 by subtracting 8 ft. from 22 ft. and using 14 ft. as the short span.

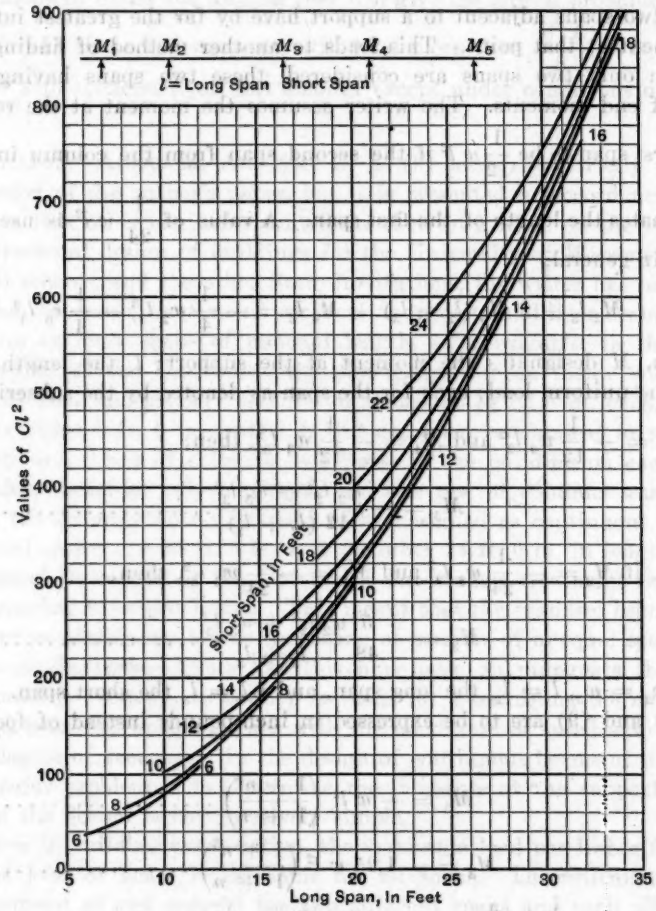

FIG. 14.

This method is approximate in that the moment at a column is influenced only by the adjacent spans and their end moments. A more accurate method is to consider four spans, two on each side of the support at which the moment is desired.

Let l_1, l_2, l_3, and l_4 represent the spans carrying uniform loads per foot of w_1, w_2, w_3, and w_4 and let M_1, M_2, M_3, M_4, and M_5 be the moments at the five supports with M_3 as the moment the value of which is desired. If the continuous beam is restrained at each end of the series of four spans the value of M_3 will be greater than it would be with zero end moments. Therefore, the beam will be assumed as fixed at M_1 and M_5. The quantity, M_1, may be readily obtained by adding an auxiliary span, l_0, carrying a load, w_0, having an end moment, M_0, each having a value of zero.

A perfectly general equation for the moment, M_3, can be obtained by applying the theorem of three moments to various pairs of spans and solving the resulting equations for M_3, which will be found to be,

$$M_3 = + \frac{a\, w_1\, l_1^3 - 2\,(1-a)\, w_2\, l_2^3 - 2\,(1-b)\, w_3\, l_3^3 + b\, w_4\, l_4^3}{8\,[l_2\,(2-a) + l_3\,(2-b)]} \quad \ldots\ldots(10)$$

In Equation (10) a and b are constants for a given case, as follows:

$$a = \frac{l_2}{1.5\, l_1 + 2\, l_2} \text{ and } b = \frac{l_3}{2\, l_3 + 1.5\, l_4}$$

For all ordinary cases Equation (10) gives a negative result because the negative terms of the numerator are usually much larger than the positive terms.

Illustrative Problem.—Find the moment at the center support of a continuous beam, in which $l_1 = 8$ ft., $l_2 = 20$ ft., $l_3 = 12$ ft., $l_4 = 15$ ft., and $w = 2\,500$ lb. per lin. ft. for all spans. If Fig. 14 is used, l_2 and l_3 only are considered giving a value of 305 for $C\,l^2$ and a moment of 305 (2 500) = — 762 500 in-lb. for M_3.

If Equation (10) is used,

$$a = \frac{20}{1.5\,(8) + 2\,(20)} = 0.385 \text{ and } b = \frac{12}{2\,(12) + 1.5\,(15)} = 0.258$$

Then,

$$M_3 = 2\,500\left(\frac{0.385\,(8)^3 - 2\,(1-0.385)\,(20)^3 - 2\,(1-0.258)\,(12)^3 + 0.258\,(15)^3}{8\,[20\,(2-0.385) + 12\,(2-0.258)]}\right)$$

$$= -\,66\,500 \text{ ft-lb.} = -\,798\,000 \text{ in-lb.}$$

Now, change l_1 to 15 ft. and l_4 to 8 ft., but leave the inner spans as before and solve for M_3 by Equation (10). Then,

$$a = \frac{20}{1.5\,(15) + 2\,(20)} = 0.320 \text{ and } b = \frac{12}{2\,(12) + 1.5\,(8)} = 0.333$$

$$M_3 = 2\,500\left(\frac{0.320\,(15)^3 - 2\,(1-0.320)\,(20)^3 - 2\,(1-0.333)\,(12)^3 + 0.333\,(8)^3}{8\,[20\,(2-0.320) + 12\,(2-0.333)]}\right)$$

$$= -\,69\,500 \text{ ft-lb.} = -\,834\,000 \text{ in-lb.}$$

In the first case the answer obtained by the use of Fig. 14 is 4.5 per cent. less than that obtained by Equation (10), while in the second case the difference is 9.5 per cent. The first error is relatively small, but the second is large enough to require correction. It should be noted that in the latter problem l_1 is less than l_2 and also that l_4 is less than l_3. Hence, the moments, M_2 and M_4, are less than those assumed in Fig. 14 and, consequently, M_3 is

greater than that given by this diagram. Therefore, it may be concluded that when l_2 exceeds l_1 and l_3 exceeds l_4 by a considerable proportion the value of M_3 obtained by using Fig. 14 should be increased by about 10 per cent. Likewise, when l_1 exceeds l_2 and l_4 exceeds l_3 by a considerable proportion the value of M_3 given by Fig. 14 may be reduced by about 10 per cent.

It should be noted that in Equation (10), different values of w can be used in the different spans, thus allowing the placement of live load and dead load on the two inner spans with dead load only on the two outer spans. In this way the maximum value of M_3 is produced.

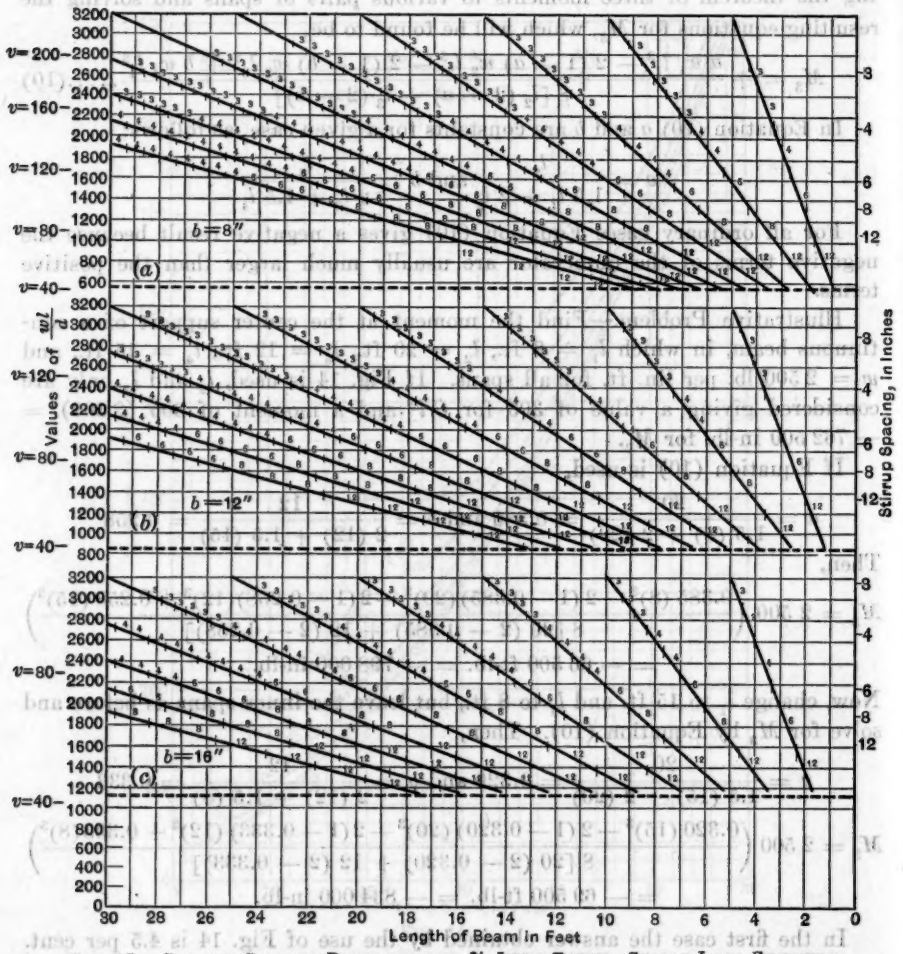

FIG. 15.—STIRRUP SPACING DIAGRAMS FOR ⅝-INCH ROUND, SINGLE-LOOP STIRRUPS, f_s EQUALS 16 000 POUNDS PER SQUARE INCH.

Stirrups.—In the design of shear reinforcement for reinforced concrete beams the writer has devised a few diagrams for his own use which may be of interest to other engineers. Here, again, he has tried to get practical results

with a maximum of speed and a high degree of accuracy. The diagrams illustrated herewith (Fig. 15, (*a*), (*b*), and (*c*)) are for three different beam widths 8, 12, and 16 in. and for $\frac{3}{8}$-in., round, single-loop stirrups at a unit stress of 16 000 lb. per sq. in., but the general principles could be used in the construction of diagrams to fit other conditions.

It is the writer's practice to choose the values of b and d for a series of beams to satisfy the maximum conditions of moment and shear and then keep these dimensions constant for the group.

For a beam carrying a uniform load per foot, the end shear, V, equals $\frac{w\,l}{2}$, part of which, $v_2\,b\,j\,d$, is taken by the concrete, and the remainder, V', by the stirrups. The stirrup spacing, s, equals $\frac{A_s f_s j\,d}{V'}$, or,

$$\frac{V'}{d} = \frac{A_s f_s j}{s} = \frac{2\ (0.1104)\ 16\,000\ (0.875)}{s} = \frac{3\,090}{s}$$

for $\frac{3}{8}$-in. round stirrups. Then,

$$\frac{V}{d} = \frac{V'}{d} + \frac{v_2\,b\,j\,d}{d}$$

or, with $v_2 = 40$ lb. per sq. in., and $j = 0.875$,

$$\frac{V}{d} = \frac{w\,l}{2\,d} = \frac{3\,090}{s} + 35\,b$$

or,

$$\frac{w\,l}{d} = \frac{6\,180}{s} + 70\,b$$

In the construction of each diagram a dotted line was drawn parallel to the base as shown in Fig. 15 (*c*) and at a distance of 70 b above it. Other parallel lines were drawn above this at a distance equal to $\frac{6\,180}{s}$ for different stirrup spacings.

In some of his earlier diagrams the writer stopped here without indicating any diagonal lines with selected stirrup spacings. A horizontal scale giving the half-span in inches was used and any particular problem was solved by finding a point in the diagram at the intersection of this length with the value of $\frac{w\,l}{d}$. This point was connected with the span center at the lower right-hand corner (see Fig. 15 (*c*)) by means of a straight edge and the required stirrup spacing was read along the straight edge.

Recently, it has been found advantageous to indicate some diagonal lines (Fig. 15) with well-chosen stirrup spacings and to use as a horizontal scale the span of the beam in feet. Obviously, no stirrups are needed beyond the point where the diagonal lines intersect the dotted horizontal line. In the region where stirrups are needed the spacing should not be allowed to exceed one-half to three-fourths the effective depth of the beam.

Illustrative Problem.—A reinforced concrete beam, 18 ft. long, having a width of 12 in. and an effective depth of 16 in., carries a load of 2 000 lb. per lin. ft. Find the spacing of ⅜-in., round, single-loop stirrups having a unit stress of 16 000 lb. per sq. in.

With a value of $\frac{w\,l}{d} = \frac{2\,000\ (18)}{16} = 2\,250$, and a beam length of 18 ft., Fig. 15 (*c*) gives the following stirrup spacing, in inches: 4, 4, 4, 4, 6, 6, 8, 8, 12, 12.

F. Theodore Mavis,* Jun. Am. Soc. C. E. (by letter).†—In his opening paragraph the author states‡ that,

"Although reinforced concrete structures have been used extensively for the past twenty-five years, and the general theory of their design has been quite fully developed, yet engineering literature, including the publications of the Society, give meager discussions of the proper distribution of reinforcing steel in concrete beams and slabs."

This charge seems to be a bit broad when one considers the valuable papers and discussions dealing with the analysis of statically indeterminate structures published by the Society from time to time. Any one of the many variations in the application of the elastic theory arrives at the moments, thrusts, and shears acting at any section by a rational method.

The author considers the case of a series of identical spans, each loaded with the same uniform load, *w* lb. per ft., and arrives at a set of rules governing the relative amounts of steel required at the center of a given span and at the supports. This is an elementary problem in mechanics, and one which should not give the structural engineer the slightest cause for worry. Almost any textbook in mechanics has a table or chart of moment coefficients at the supports for a structure of this kind having from two to six or more spans.

All this is of no value to the structural engineer who is dealing (1) with girders carrying beam loads; (2) with frames made up of beams or girders which are not of equal length; (3) with structures carrying a uniform load in one span differing from that in another; (4) with structures carrying machinery or moving concentrated loads; or (5) with structures having haunched beams or girders. Handbooks and textbooks on reinforced concrete contain much more information of value in the design of even ordinary structures than that presented in the paper.

The author states§ that the total reinforcement required by the formula, $M = \frac{1}{8}\,w\,l^2$, may be divided in any manner between positive steel at the center and negative steel at the supports. The writer assumes that a monolithic concrete structure built in accordance with a satisfactory design is one that will not develop serious cracks when it is subjected to the loads for which

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† Received by the Secretary, September 16, 1926.

‡ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1106.

§ *Loc. cit.*, p. 1109.

it was designed. Consequently, he can hardly agree that placing all the steel in the top or all the steel in the bottom of a beam continuous over a support would make even a "theoretically satisfactory design".

In the design of a continuous framed structure the engineer is faced with a problem similar to the one which is met in every structural design: (1) He must know the dimensions of every part of the structure before he can analyze it; and (2) he must analyze the structure before he can design the sections.

Assume that the designer has selected the sizes of members in the structure, based on preliminary calculations and on his experience and judgment. This first draft of the design should, for economy of time and labor, approximate the final design. Now, assuming that the structure "hangs together", he can make an analysis of it under critical conditions of loading, and determine the moments, thrusts, and shears acting at any section. As long as the structure behaves as an unbroken elastic body, these moments, thrusts, and shears obtain, regardless of the amount or location of steel reinforcement. Knowing the internal forces at any section, the engineer can design that section to resist these forces, placing the required amounts of steel in the various members to "take" tension and prevent cracks from opening. Evidently, the author does not question the conventional methods for calculating the amount of steel required in a given section when the bending moments and shears are known.

Methods of analyzing a statically indeterminate frame are well known and an elastic analysis of an important structure takes a relatively short time. Even a rough analysis would give a basis of design which is much more reliable and more widely applicable than the restricted set of rules laid down by the author.

The author further states* that the bending produced in a column is a question of the amount of negative reinforcement in the beams which frame into that column. In what way does the bending moment in the columns depend on the amount of steel in the top of the beams, provided cracks do not open in the structure? As long as cracks do not occur, the bending moment at a given section depends only on the loads and the elastic properties of the structure, and will be the same for a given loading regardless of the amount of reinforcement in the top of the beam at a support. In this case also, would it not be better to analyze the structure as an unbroken elastic body and design the members to resist the internal stresses determined in such an analysis, placing steel where necessary to "take" tension?

Rules such as those laid down by the author are dangerous in the extreme. Unless an engineer is qualified to analyze a structure to carry the loads coming on it he had better not quibble over the difference between the steel required by such formulas as $M = \frac{1}{8} w l^2$ and $M = \frac{1}{6} w l^2$. In such a case it is better that he "play safe", or else leave indeterminate structures, which include monolithic concrete structures, alone.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1111.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).†—Doubtless, in the great majority of concrete designs not enough attention is paid to the proper distribution as well as amount of reinforcing steel. The complete analysis of a reinforced concrete structure, even so simple a member as a beam, is so tedious and complicated that the use of several empirical simplifications has become practically universal. So common has this become that only a few of the many who use standard methods of reinforced concrete design realize that there are approximations inherent in the formulas; and far too many know little of the principles involved.

Careful analysis of a reinforced concrete design almost invariably points to possibilities of large savings in reinforcement. At the same time, the amount of work and time required for such analysis and the additional field cost to follow special and unusual details make it (the writer believes) in the majority of designs uneconomical for the owner and contractor to deviate from present practice.

The writer recalls one instance which will well illustrate this belief. In detailing the steel for a flat-slab floor, it is customary to continue all panel and column band rods (in the top and the bottom of the slab) past the line of inflection a distance at least equal to that required to develop the full tension value of the rod by bond. This is equivalent to the requirement that the full value of all rods be required on each side of the line of inflection; in other words, that the full positive and negative bending moments be taken care of at opposite sides of each line of inflection. This is evidently too conservative an assumption. By cutting each rod 12 in. shorter and shifting alternate rods right and left of the line of inflection by 6 in., a saving of more than 5% in weight of reinforcing steel is effected; and at each line of inflection one-half the maximum moments can be safely taken care of—certainly sufficient for a point of zero theoretical moment. To allow for a variation in location of the lines of inflection, due to non-symmetrical loading or other causes, the bends in the rods are staggered alternately 6 in. each side of the lines of inflection. In this manner, all rods are identical as to lengths and locations of bends—an ideal condition for the project for which all rods are ordered bent to shape or bent on the bench.

However, as is often the case, the band rods are first placed in position on the forms and bent into shape *in situ*. Considerable extra time and work are then required to place the rods in the proper staggered position and to stagger the bends. It is doubtful whether the saving in steel compensates for the extra labor. The writer, therefore, only uses this refinement in detail where the size of rods is so heavy that the bending must be done at the bench or when he is certain that bent rods will be ordered for the job.

Mr. Myers' analysis of maximum bending moments in the section entitled "General Principles"‡ is in error because he has assumed that the elastic curve of a continuous or restrained beam can be cut into sections which are identical

* Cons. Engr., New York, N. Y.

† Received by the Secretary, September 21, 1926.

‡ *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1106.

with the elastic curves of simple and cantilever spans. That this is not the case can be seen from the following analysis.

In an elastic structure, such as a beam, the curve of deformation must be continuous and of the second degree. There can be no cusp points, that is, at each point there can be only one slope to the curve. In Mr. Myers' solution, the slope of the "simple span" at the point of inflection is not the same as the slope of the "cantilever" at the same point.

The simple span, in the general case shown in the Fig. 16 (*b*), of Span $2s$, end reaction of $w\,s$, has a slope at the end of $-\frac{w\,s^3}{3\,E\,I}$. In order that the curve of the continuous beam be smooth, this slope must be equal and opposite in sign to the slope at the end of the cantilever, shown in Fig. 16 (*c*). The latter slope is $\frac{w\,c^3}{3\,E\,I} + \frac{s\,w\,c^2}{2\,E\,I}$.

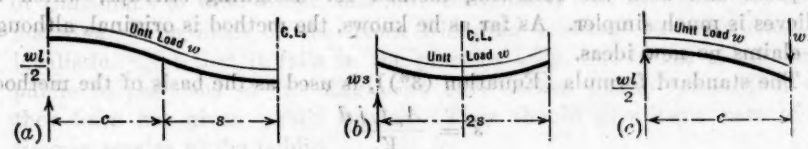

FIG. 16.

Equating and solving:

$$\frac{s^3}{c^3} - \frac{3}{2}\frac{s}{c} - l = 0$$

or,

$$s = \frac{3}{2}\,c \text{ (approx.)};\ c = 0.2\ l;\ \text{and } s = 0.3\ l.$$

The approximate solution is about 2% in error. Only for the case of $c = 0.2\ l$, that is, when the point of inflection is at the 0.2 point of the span, is Mr. Myers' assumption correct.

To check this solution, the deflections of the beam can be investigated. A continuous beam has a maximum deflection of $\frac{w\,l^4}{384\,E\,I}$. The deflection of the center simple span ($0.6\,l$) is:

$$\frac{5\,(0.6)^4\,w\,l^4}{384\,E\,I}$$

and of the cantilever of Span $0.2\,l$, loaded uniformly is:

$$\frac{(0.2)^4\,w\,l^4}{8\,E\,I}$$

and of the cantilever loaded with the reaction of the simple span as a concentration at the free end is:

$$\frac{(0.3)\,(0.2)^3\,w\,l^4}{3\,E\,I}$$

Summing the three deflections (this is permissible if all the strains are within the elastic limit), the total deflection is:

$$\frac{215\ w\ l^4}{80\,000\ E\ I}, \text{ or } \frac{w\ l^4}{377\ E\ I}$$

so that the approximate location of the point of inflection at the 0.2 span point gives a deflection only $\frac{7}{384}$, or less than 2% in error. This is the basis of the empirical rule for the location of bends in reinforcement of continuous beams and slabs, including flat slabs, at the 0.2 point of the spans.

The entire analysis of Mr. Myers falls down on the removal of the false assumption of beam action.

The method described in the paper for the determination of the number and spacing of stirrups is so long and tedious that it can hardly become of common use. The writer has developed some short cuts in concrete design for his own purposes and uses the following method for designing stirrups, which he believes is much simpler. As far as he knows, the method is original, although he claims no new ideas.

The standard formula (Equation (3*)), is used as the basis of the method:

$$s = \frac{A_v f_s j\ d}{V_1}$$

But,

$$V_1 = (v - v_2)\ b\ j\ d$$

in which, v is the unit shear.

Hence,

$$s = \frac{A_v f_s\ j\ d}{(v - v_2)\ b\ j\ d} = \frac{A_v f_s}{(v - v_2)\ b}$$

a simple and easily remembered as well as extremely useful form. There are a limited number of stirrup rod sizes; usually only $\frac{3}{8}$ and $\frac{1}{2}$-in. round rods are used. According to any one building code, the value of f_s is constant, and the concrete is assumed to take care of a certain unit shear.

For a $\frac{3}{8}$-in. rod, A_v is 2×0.110, f_s is 16 000, and v_2 is 40. By substituting, there results:

$$s = \frac{440}{v - 40}$$

From a shear diagram or from mental calculations, the v at any point in the beam can be found; then by slipping the slide of the slide-rule, with the indicator at 440, the various successive stirrup spacings are read directly. It is well to remember that a formula giving the total number of stirrups required cannot take into account the requirement that the maximum stirrup spacing is not to exceed some fraction of the beam depth, say, one-half or three-quarters (the usual values specified). A simple straight-line diagram can be used to determine the part of each span, for any definite end shear, which must be reinforced with stirrups.

* *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1118.

THE ENGINEER AND THE TOWN PLAN

Discussion*

By Messrs. Noulan Cauchon, Morris Knowles, and William T. Lyle.

Noulan Cauchon,† Esq.—The speaker appreciates the lively interest that the American engineer has taken in town planning. In Canada we have not, so far, been fortunate enough to interest the Engineering Profession as a whole in it. It is hoped that these discussions will assist in opening the eyes of Canadians to what Americans have recognized, that is, the future of town planning. The ideas of the Town Planning Institute are considered very idealistic. Not that it fails in the practical side, but it has to educate the public to the need of it. Engineers have lacked public recognition where they have not given public service. They should give some part of their time in service to the public.

The Town Planning Institute was founded in Canada on the principle that all men coming to it have to be recognized by their own professions before it admits them. In other words, it is sort of a "Post-Graduate Institute" and is based on the principle that there be no overlap of function. It wants the architect to know enough about engineering not to put a building where he can see that a street will have to be widened, and for an engineer to know sufficient of architecture and art to restrain him from mutilating beauty, existing and potential. It desires to have an overlap of knowledge, but not an overlap of function.

Town planning should be based on a biological aspect. A city in which the streets are narrow and dark does not provide a biological condition fit for the maintenance of life. It is simply a problem of translating that view into engineering terms. The engineer who is planning the city is a biologist and has to know and plan for the conditions under which life can exist and thrive. It is not new as knowledge, but is simply a focusing of science to the purpose of human betterment—in other words, human efficiency. The congestion of streets is simply a cancer in the organism of the city.

Business men have said, "Why do you not let the city grow naturally"? It cannot grow naturally because it is a machine in which to live. What will happen unless the engineer designs the city so that it will not go to pieces? A city expands by accretion merely, being devoid of inherent biological balance and checks.

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* This discussion (of the paper by James Ewing, Esq., presented at the meeting of the City Planning Division, Montreal, Que., Canada, October 15, 1925, and published in September, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr.; Past-Pres., Town Planning Inst. of Canada; Director, Am. City Planning Inst. and City Planning Conference; Chairman, Town Planning Comm., Ottawa, Ont., Canada.

The question of highways is an important issue. Canada is over-capitalized in railroads. The United States has the same problem, but it can carry it better because it has a bigger population. Every one has gone crazy on the subject of highways. Let us not run away with the idea that we can build an unlimited number of highways, or an over-capitalization in highways, as in railways, will result. Instead, as an engineering policy, the effort should be toward intensive cultivation and population along those highways to sustain their cost. It is not a political question but an economic one. As it is, the cities are carrying the burden. This is simply a thought for engineers.

Another important thing in respect to highways is to obtain and preserve ruling grades on them. The first difficulty in a grade separation is that the railway wants to retain its ruling grade and will give good reasons for so doing. The point is that both the railway and the highway should have and preserve their ruling grades and divide the cost.

Mr. Alvan Macauley, head of the Packard Motor Car Company, Detroit, Mich., is the first big automotive executive who has recognized that town planning is the only solution to the problem of the highway and its congestion. His Company has issued two interesting pamphlets on the subject. Now he advises all motor companies and all motor agents to exert their influence toward having town planning commissions established.

Morris Knowles,* M. Am. Soc. C. E.—Mr. Ewing's paper is so broad in scope that one hardly knows what to select that will be of greatest interest to discuss. The author has been frank, with usual English bruskness, in telling engineers of their faults and failings. Much of it is truth they need to know. He is not less veracious than he is forceful in his reference to the "brave, brawny and brainy fellows", who are wonders in producing "monuments of engineering enterprise and skill". These may be "lasting memorials of pitiful judgment and radically bad planning"; and—as he says again—the work of "splendid engineers but altogether indifferent planners".

Another discussor suggests reasons why other professions succeed in having public influence. Probably he did not intend to imply, and most engineers will not agree, that in order to have influence it is necessary to be servile. Honest, efficient service may be performed without fawning. The reason for lack of influence lies deeper. The engineer is not so gregarious as many other types of mankind. He is accustomed to deal with material things more often than with human beings. While he frequently works in groups, his labors and discussions are objective rather than subjective; they deal with the stress of concrete beams rather than any stress of public and human affairs.

This is somewhat different from the other professions. The clergyman, the lawyer, and the doctor frequently gather together and discuss public questions quite as often as professional matters. They sometimes settle important public affairs without having definite knowledge. The engineer is prone to discuss the technicalities of a thing—its detail rather than its conception and planning and its effect on other public activities. In so far as they have

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

this lessened public contact with each other and with other groups, and participate less in the discussion of political and public questions, the engineers' influence is less than that of other professions.

Engineers will agree that the basis of city planning is engineering, but certainly they can have more influence in the community if they can "put over this idea" to others—can cause them to realize that the fundamental backbone of any planning, before the design or execution, is the acquirement of information of fundamental engineering nature, such as topography, distances, elevations, mapping of natural features, etc. Of course, the acquirement of this information is not designing, but one must plan a place for people to live, a location for stores in which to do business, and industries in which to work, with suitable surroundings and facilities for each. Adequate highways of sufficient width for convenience and transportation and the relative placing of workshops, stores, and homes in suitable locations, without jarring juxtaposition on each other, is primarily an engineering study. He who has been doing this from time immemorial is the City Engineer. True, some of it has been badly done but much of it has been done so well as to make water systems, sewer systems, highways, and other forms of public utilities planned for the future rather than the immediate present. Thus it may be said without fear of contradiction that the original city planner was the early and original city engineer.

Surely it will be worth while to remember this, to preach this; yes, but to execute better. At the same time engineers have much to learn from the kindred professions in the line of conception, in design, in harmony, and in execution, and likewise much to learn as to the advantage of public discussion, preachment of their ideas, and selling of their services. They may then extend their influence and make their contribution more worth while. In other words, engineers may be of service without being servile.

WILLIAM T. LYLE,* M. AM. SOC. C. E. (by letter).†—It is to be regretted that certain papers published by the Society are read only by those attracted by the titles. The paper by Mr. Ewing will interest all engineers engaged in city planning; but by reason of breadth of vision, keen criticism, and constructive wisdom it deserves to be read also by all other members.

The Civil Engineering Profession during the last century has passed through a cycle of development. As differentiated from Military Engineering it concerned itself with civil pursuits. The earlier civil engineers were engaged in the projecting, designing, and constructing of railroads, canals, power plants, harbors, bridges, and many other public works. Later, came the specialized subdivisions mentioned in the paper, some at first little higher than craftsmanship. The lowly origin of Sanitary Engineering is within the memory of many engineers still living, and articulated bridge building is known to have had its beginnings in carpentry. An examination of the files of the technical press forty years ago is both instructive and amusing.

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.
† Received by the Secretary, September 18, 1926.

To-day, the drift is toward higher standards and greater usefulness and, strange as it may seem, the city planning and regional planning engineer is pointing the way. Engineers must be more than mere technicians; they cannot in self-respect be content to follow without question the direction of those who employ them; they must exercise a projecting and guiding control in every plan. When they have thus asserted themselves in determining the feasibility of a project as well as the design of the works, they rise to the level and dignity of what might be called planning engineers, for want of a better designation; in other words they become engineers in the highest possible sense.

Thus, although highly specialized subdivisions will always be necessary, a strong current is running in an opposite direction, that is, toward unification. Engineers of broader vision and enlarged power are indispensable to the great works of modern civilization—works which are intimately related to many others and which cannot be properly planned without a comprehensive and far-seeing vision.

GRIT CHAMBER PRACTICE

A SYMPOSIUM

Discussion*

By Messrs. A. F. Macallum and Kenneth Allen.

A. F. Macallum,† M. Am. Soc. C. E.—Apparently, little attention has been given to the design of the concrete grit chamber. No doubt the reason is that grit chambers have not always been considered necessary with different types of sewage disposal plants except the activated sludge treatment. Another point that has not been discussed in this Symposium is the character of road surface from which the sewers bring the surface drainage to the disposal works, which will affect the size of these chambers, or even the necessity for them. When the surface drainage was over earth, gravel, and macadam roads, such as were common years ago in many suburban areas, there was a great quantity of road detritus carried to the disposal works and in many cases grit chambers were desirable if not necessary. Since then, in nearly every city, roads have been treated with asphalts or tars so that the road material carried away in the sewers is much less than before, and the disposal works is able to handle this without the necessity of a grit chamber.

However, with the development of the treatment of activated sludge it is evident, according to the Symposium, that the grit chambers are once more appearing in the picture.

Kenneth Allen,‡ M. Am. Soc. C. E. (by letter).§—The attempt to secure a reasonably clean grit by the control of velocities in the grit chamber is a modern development. The detritus tank as generally found in England and exemplified in Germany at the Hamburg screening stations, and in Paris, France, at the Clichy Pumping Station, is not designed to attain this object, being practically a sedimentation tank having a brief detention period. In America the two deposit sewers, 16 ft. high by 8 ft. wide and ¼ mile long, built more than 40 years ago in connection with the main drainage works of Boston, Mass., afford an example of the same class. These are placed after the pumping station, their function being to prevent deposits in a long tunnel siphon under Dorchester Bay.

* This discussion (of the Symposium on Grit Chamber Practice, presented at the meeting of the Sanitary Engineering Division, Montreal, Que., Canada, October 15, 1925, and published in September, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Commr. of Works, Ottawa, Ont., Canada.

‡ San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

§ Received by the Secretary, September 14, 1926.

Modern German and American practice attempts to secure as complete a separation of the inorganic matter as practicable and the experiments carried out at Cleveland, Ohio, by Mr. Gascoigne have aided greatly in providing a scientific basis for design.

The experience at Cleveland with vanes separating the upper "flowing through" portions of the grit chamber from a lower compartment receiving the grit was similar to that at the Dyckman Street screening plant in the Borough of Manhattan, New York City, where the resulting deposit contained a large quantity of organic matter.

An interesting attempt to maintain a fairly uniform velocity is illustrated in the design of the grit chamber of the Frankford Creek Intercepting Sewer, Philadelphia, Pa., where a pair of vertical vanes in the chamber are pivoted on axles at each side. The vanes are of such shape that when closed there remains a bell-shaped opening for the low flows. As the rates increase the leaves are opened until, at times of maximum flow, the entire cross-section of the chamber is available.

For large installations, where mechanical means for cleaning are justified, the use of hoppers or sumps appears to have advantages over the simpler and less expensive flat-bottomed type. In such installations the writer believes that cleaning by a grab-bucket is more reliable and, in the end, more economical than by the use of compressed air or bucket elevators. It is hoped that others with a larger experience in this phase of sewage works operation will take this opportunity to furnish more specific information.

PRODUCING CONCRETE OF UNIFORM QUALITY

Discussion*

By T. P. Watson, M. Am. Soc. C. E.

T. P. Watson,† M. Am. Soc. C. E.—Engineers and correlative interests are greatly indebted to Mr. Young for the presentation of this comprehensive paper on a much mooted subject. The pioneering of Mr. Young and his Canadian associates in the large-scale production of "uniform quality concrete" is worthy of unstinted praise and is an excellent example of what can be accomplished.

A study of the methods explained in the paper, and their comparison with general practice, impresses those who have been familiar with concrete production during the past twenty years, that the art of manufacturing concrete has not kept pace with the use of this valuable construction material, and that the transition from "placing" concrete to "pouring" it passed without sufficient attention being given to the use of excessive quantities of mixing water.

The use of excessive quantities of water, to take advantage of mechanical and other means to decrease labor costs and to meet the urge of the maximum possible production, continued until the actual failure of concrete structures awakened the dormant allied interests to the seriousness of the situation.

The views being put forth for the production of better quality and uniform concrete mixtures are usually not new, but are rather a revival and application to present-day requirements of the theories presented prior to 1900 by various investigators whose warnings were passed unheeded or forgotten.

The water-cement ratio theory of strength was proven by tests‡ thirty years ago, and the theory of proper proportions—that the voids in the coarse aggregates should always be filled with mortar—was presented about the same time. Practically every book on the subject of concrete has called attention to the laxity of using arbitrary loose proportions of aggregates regardless of their sizes and shapes.

In spite of the discussion of this subject and the publicity given to the active demonstration of Mr. Young and his associates, as well as many other reputable concrete users, that the accurate control of concrete manufacture is both practical and economical, the majority of concrete operations are

* This discussion (of the paper by Roderick B. Young, Esq., presented at the meeting of the Structural Division, Montreal, Que., Canada, October 15, 1925, and published in September, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Engr., Eng. Dept., P. R. R., Pittsburgh, Pa.

‡ Report of the New York State Engr. and Surv., the late George W. Rafter, M. Am. Soc. C. E., 1897, pp. 375–460.

continuing in the use of haphazard and careless methods, disregarding water content and a rational proportioning of aggregates, even although casual observation will prove that many of these structures exposed to northern climatic conditions are almost certain to decay in a comparatively short time.

A glaring reflection on the Engineering Profession, as regards concrete manufacture, is the apparent lack of comprehension of the function of competent field supervision. As long as engineers fail to realize that adequate and competent field supervision and inspection are necessary to control the handling of the product from the time of proportioning the aggregates until the concrete is in place and cured, the problem of producing "uniform quality concrete" will not be solved.

Engineers can advantageously ponder Mr. Young's admirable concluding formula:*

"Controlled concrete = suitable materials + a bit of theory applied with common sense + good workmanship + competent supervision—nothing more and nothing less".

* *Proceedings*, Am. Soc. C. E., September, 1926, Papers and Discussions, p. 1405.

WATER-RATIO SPECIFICATION FOR CONCRETE

Discussion*

By Messrs. C. R. F. Coutlee and A. S. Woodle, Jr.

C. R. F. Coutlee,† M. Am. Soc. C. E.—In the fall of 1886 the speaker first saw concrete placed. It was in the bottom of a small bridge abutment in Beaudette River. First, 12 by 12-in. timbers were laid 1 ft. apart and then some natural cement concrete was placed between them. On top of these, other timbers were laid crosswise, and the spaces similarly filled; the quantity of water was not excessive. The timber was considered necessary to give strength to the concrete.

In the Nineties more than 200 000 cu. yd. of concrete were placed in the Soulanges Canal works. Absolutely the greatest quantity of water that could be put into it was used, and it is still standing, in fact, it has been referred to as among the best examples of concrete. Of course, a porous sandstone was used, which absorbed water freely and that must have corrected the water-cement ratio. Only mass concrete was used, the weight being taken at 125 lb. per cu. ft.

The sand was measured without regard to moisture and 10% more than the voids of the stone was used. It was clean, of fairly large and strong grain. European cements, mostly Belgian, were used in Canada then; at present, there is one big cement company in the country, and the product is, like Caesar's wife, beyond suspicion.

In placing concrete under water for walls on top of cribwork, the general practice was to let it set in the skips for an hour or so. The area was divided by false forms, and the skip emptied slowly to form an end dump. Canvas aprons were tacked to the forms if there was wave action.

A. S. Woodle, Jr.,‡ M. Am. Soc. C. E. (by letter).§—The chief difficulty in obtaining general recognition and adoption by engineers of the scientific method of manufacturing concrete as proposed by Professor Abrams in his water-ratio theory, has been that of educating the profession in the theory sufficiently to convince its members of the validity of the tests and the soundness of the conclusions made from them. This recognition has been gradually brought about by the frequent discussion in the technical press, at meetings

* This discussion (of the paper by F. R. McMillan, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., presented at the meeting of the Structural Division, Montreal, Que., Canada, October 15, 1925, and published in September, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Ottawa, Ont., Canada.

‡ Engr. of Plant, Baldwin Locomotive Works, Philadelphia, Pa.

§ Received by the Secretary, September 11, 1926.

of technical societies, and by aggressive campaigning on the part of the Portland Cement Association, until it has become the exception rather than the rule to see a concrete job of any magnitude built by the old-fashioned slipshod "rule-of-thumb" methods.

There has, however, been one deterrent condition in effect, which has retarded this desirable movement, that is, the lack of a standard specification by which ordinary concrete mixes in every-day use may be proportioned without the assistance of a concrete testing laboratory. In the writer's opinion, the specifications submitted by the authors are a splendid first approximation to such a standard specification; in fact, if they were extended to include both weaker and stronger mixes than those shown, there would be little left to criticise.

A table giving the slump to be expected from various combinations of water-cement ratio to coarse and fine aggregate ratio would be very useful. Also, the specifications would be more complete if mention were made of admixtures, particularly calcium chloride, which seems to be coming into favor as an agent for curing the concrete, and for improving the strength at all ages.

One stumbling block which crops out several times in this paper, and against which every student of concrete proportioning has stubbed his toe, is the need for a clear-cut scientific definition of, and test for, the very elusive property, "workability". He who successfully evaluates this will o' the wisp variable—which is a function of the water ratio, of the cement aggregate ratio, of the fine aggregate-coarse aggregate ratio, of the fineness modulus of the aggregate (both coarse and fine), probably also of the shape and texture of the aggregate, and perhaps of many others—will have taken the next great step forward in the scientific manufacture of concrete.

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED

Discussion*

By Messrs. John W. Alvord and William T. Lyle.

John W. Alvord,† M. Am. Soc. C. E. (by letter).‡—One cannot read Mr. Nolen's inspiring address without broadening one's conception of professional service to the public, and receiving, as well, a stimulating vision of the fuller possibilities of community life.

Whatever prevents men from unnecessarily crowding, interfering, and distressing one another and frees them from hindrances to the better expression of their varied and constructive activities, and, finally, inspires them with ideals of individual and community welfare, is, indeed, a great forward movement.

Thus, the writer would define town and city planning as those foresights in physical adjustment necessary for the fullest expression of the individual in his community life.

The Engineering, Architectural, and Landscape Engineering Professions, as well as other kindred fields of "practical artistry" and specialized service, have a common aim and purpose in uplifting service to the public; and it is singular that engineers, the professional descendants of Leonardo da Vinci, Michel Angelo, and many others, who were at once both painters, sculptors, architects, and engineers, should have grown to think of these fields as separated callings, to their distinct disadvantage, when as a matter of fact they are only different and overlapping aspects of the one broad field of creative design.

This was made amusingly evident some years ago when the architects and engineers attempted to define their several callings for the purpose of legal regulation. It simply could not be done.

The work of successful city planning, as Mr. Nolen has shown, requires a broad insight into every class of creative design, and foresight from the intensely practical up to the artistic and spiritual. It calls for broad-minded men, not only with especial kinds of training, but having above all sympathetic insight into all other fields of human endeavor, besides the one in which they have been especially trained. Specialization tends to obscure these broader requirements, and it goes without saying that to overcome its narrowing con-

* This discussion (of the paper by John Nolen, Esq., presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926, and published in October, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (Alvord, Burdick & Howson), Chicago, Ill.

‡ Received by the Secretary, January 23, 1926.

centration is a cultural success that is attained by but few men who, with great diligence, make the attempt to meet these requirements.

It was the writer's good fortune, during the World War, to spend a year with the United States Housing Corporation at Washington, D. C., where town planners, architects, and engineers were closely co-ordinated in nearly one hundred housing developments. Early in that experience the impression on the writer was, not the separateness of these three callings, but, at bottom, the remarkable similarity. One could hardly call them partly overlapping, in any broad review at least. They were, perhaps, best described as uni-central in purpose, with varying aspects along their outer boundaries, or in matters of detail function.

Unity of general purpose and fitness of each co-ordinating function, with variety and harmony in detailed method, as practical opportunity is afforded, is the keynote of good town planning. Mr. Nolen has admirably pointed out some of the necessary details for the application of these principles.

WILLIAM T. LYLE,* M. AM. SOC. C. E. (by letter).†—"Town Planning and Its Relation to the Professions Involved" is undergoing a great change, owing to the changing nature of towns themselves. This development in its regional character requires the solution of new problems, and emphasizes the desirability of uniting the professions in developing a city planner.

Were the problem one of *de novo* design as in the case of Washington, D. C., the city planner with training as a landscape architect would be far better qualified, other things being equal, than the planner from the ranks of architects or engineers, even if he would find it necessary to consult with the others.

There was a time, not many years ago, when city planning was concerned with the laying out of the "city beautiful"—when the design and construction of an elaborate park system was supposed to be the chief element in municipal improvement. There was also a time, even more recently, when city planning consisted principally in the building of a showy civic center.

Those days have passed. People now realize the magnitude and comprehensiveness of a task which involves all the physical elements of the city's conformation, not only of the city proper, but also of the region about it from which it draws for its needs and to which it ministers as a center of supply and distribution.

The shifting emphasis has been brought about by the change in the towns and cities themselves. Towns laid out for the conditions of Colonial times, with streets of more than ample width for the requirements of the Eighteenth and even the Nineteenth Century, have grown to so great a degree that those same streets are inadequate to the demands of even a single office building. Railroads once properly located on the outskirts of small towns now find themselves in the very heart of large cities with grade crossings seriously menacing public safety, and seriously hampered in operation by reduced speed of trains and increased expense in the protection of crossings. Shipping facilities

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† Received by the Secretary, July 21, 1926.

once equal to every demand made on them now find themselves hampered by restricted pierhead and bulkhead lines and by inadequate railroad terminal co-operation. In many cities the housing problem is acute; in many, also, the problem of local passenger transportation remains unsolved; and in all the problem of street traffic is one of the greatest difficulty.

Owing to the growth and development of towns and cities the importance of the application of the engineer's art in the making of the city plan is becoming vastly more important. It is no longer sufficient for the city planner to confine his attention to the laying out of streets, parks, and civic centers. He must also consider problems of traffic control which have baffled solution; problems of how and to what extent streets can be widened; problems of river and harbor design and improvement; problems of terminals and bridges, together with their approaches; problems of track elevation and depression, trackage elimination, union stations, loop lines, and classification yards.

City planning is intensely urgent and practical, and the author has pointed out the demand for talents of great versatility. All these talents have not been found in one profession—not in the civil engineer, certainly, the architect, or the landscape architect. With the shifting of emphasis, from the esthetic to the architectural, and lastly to the utilitarian, such talents as are found in the engineer are coming more and more into demand.

The time has arrived for the development of the city planning engineer, a man in whom the great work of city planning can "head up" and under whose regulation and co-ordinating guidance the great and diversified work can proceed.

He must be primarily a man of the engineering type, capable of solving those pressing modern problems which are baffling solution. He must be a man trained in co-ordinative architecture so far as it pertains to the grouping and arrangement of buildings, in street design, and in the general principles of park design; but above all in those economic and constructional problems due to restriction and congestion in traffic and transportation. Such planners have not yet been developed, except possibly a few in the school of practical experience.

In view of the great demand for broad-gauge men of this type for the solution of existing difficulties in congested cities, it is time for a few of the larger universities to turn their attention to the development of engineers of this type, men not only versed in the general principles of city planning architecture but qualified to plan for the relief of modern congestion on the firm practical basis of economic municipal wisdom.

GARBAGE DISPOSAL
A SYMPOSIUM

Discussion*

By Messrs. Walter A. Sperry and Kenneth Allen.

Walter A. Sperry,† Esq. (by letter).‡—Prior to 1908 the garbage of Grand Rapids, Mich., was burned in an incinerator. At present (January, 1926), the incinerator is used to burn trash and the carcasses of animals. In 1924-25, 6 315 loads of refuse, and 8 700 odd animals, and birds, were destroyed. It is possible that a boiler may be operated in conjunction with this plant to provide heat during the winter to a group of utility buildings, shops, and garages for the use of the Service Department.

Beginning with 1908, the City of Grand Rapids adopted the plan of contracting with private parties for the disposal of its garbage by hog feeding. From 1908 to 1925 three or four companies have successively taken over this business, and at present one firm has a contract which will expire in two years. This method of disposal has been quite satisfactory, and, no doubt, will be continued. The general provisions of the contract, as it is now written, are as follows.

The City collects the garbage and delivers it to a central loading point, where it is transferred from the collection tanks to steel cars of the gondola type, eight of which are owned by the City. The refuse is then transported to the pig-feeding farms, about forty-five miles northwest of the city. For this the City receives payment at the rate of $50 per month for the total garbage received, and, in addition, receives from the contractor $150 per year toward the cost of maintaining the railroad equipment. In the latest contract the City agrees to contribute toward the freight at the rate of $12 per car, the freight having been adjusted on a car basis in place of a tonnage basis, so that there are no restrictions on loading cars to capacity, the contractor agreeing to dispose of all garbage regardless of its amount. The contractor further agrees to keep the cars in a clean and sanitary condition, and the City agrees to protect the contractor's rights to all garbage accumulating within the city limits. Finally, the City is protected by a bond of $10 000 for the faithful performance of the contract.

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* This discussion (of the Symposium on Garbage Disposal, presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 21, 1926, and published in October, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Director of Public Service, Grand Rapids, Mich.

‡ Received by the Secretary, January 30, 1926.

The City has no control over the final disposal of the garbage. It is understood that the contractor feeds 2 100 to 2 500 pigs through the season. The garbage, on receipt at the farm, is loaded into wagons and transported to the feeding lots, where it is scattered on the ground.

The budget for this service within the city amounts to about $80 000. This sum provides for a force of 35 to 45 men, and a fleet of 19 collection trucks. Most of the trucks are equipped with flat platforms, and each truck accommodates two steel tanks with covers. The tanks are 10 ft. long, 2 ft. deep, and 4 ft. wide, with trunnions at each end for unloading. At the unloading dock an overhead motor-driven crane is used to lift the tanks and empty the contents into the cars by means of a reversing bridle.

During 1925, the use of horses for garbage collection was discontinued. Furthermore, motor trucks are more desirable than tractor-drawn trailers, on account of their superiority in maneuvering in and out of alleys. Tractor-drawn equipment has been found unsafe in the hill districts. For these reasons the Department has standardized more or less on motor-truck equipment.

During 1923 and 1924, an average of about 1 400 tons of garbage per month was collected, the maximum tonnage occurring in September (2 000 tons) and the minimum in February.

The net cost of operating the Department was $73 431 in 1923, and $74 712 in 1924. In each year revenues of about $3 700 were received. The net cost per ton was $4.33 in 1923, and $4.46 in 1924, representing a net cost per capita of $0.49 in 1923, and $0.47 in 1924.

The Superintendent for this Division is paid a salary of $2 100 per year, and the truck drivers and helpers are paid at the rate of $5 per day; after six months' employment at $4.50 per day.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).†—In the City of New York, N. Y., the separation of garbage, rubbish, and ashes and the disposal of refuse in the Boroughs of Manhattan, The Bronx, and Brooklyn, is under the jurisdiction of the Department of Street Cleaning. In the Boroughs of Queens and Richmond refuse disposal is under the Bureau of Street Cleaning of the Borough. Ashes are disposed of separately in Queens, but in Richmond no separation is required.

In 1924, of the total refuse produced, 1 729 213 cu. yd. of garbage and 3 224 486 cu. yd. of ashes were dumped at sea and 3 840 248 cu. yd. of refuse, mostly garbage and rubbish, with 5 941 tons of solid trade wastes, were incinerated. The total refuse includes, in addition, 3 949 545 cu. yd. of ashes and rubbish taken to Riker's Island and other large quantities, including some garbage, disposed of on inland dumps, and of ashes used in grading streets.

The disposal of garbage at sea has been on such a scale that a brief statement of quantities, methods, and costs involved may not be amiss.

Disposal by Dumping at Sea.—The disposal of garbage and ashes by dumping at sea is probably carried on by the City of New York on a larger scale than elsewhere. Owing to the abandonment of a reduction plant on

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.
† Received by the Secretary, March 12, 1926.

Staten Island in October, 1918, it became necessary to revert to the disposal of garbage at sea as had been the practice prior to 1896, and, again, as an emergency measure due to the burning of the Barren Island reduction plant in 1906. In 1924, 1 138 952 cu. yd. of garbage and 1 051 335 cu. yd. of ashes from Manhattan and The Bronx and 590 261 cu. yd. of garbage from Brooklyn, were disposed of in this manner.

The operation is as follows: Ashes and garbage (except condemned foodstuffs) are collected and brought to twenty-two water-front dumps (fifteen of which are for ashes only) where the material is deposited in scows or "dumpers" holding about 1 800 cu. yd., or 900 tons. Of these there are twenty now in service: Six bottom dumpers, to replace which would cost about $62 500 each, two old side dumpers costing $50 000 each, and twelve new side dumpers, costing $45 000 each. The bottom dumpers are 134 by 34 ft. and the side dumpers 137 by 34 ft., in plan, with sides 14 ft. high. Each dumper contains eight compartments or pockets, which are emptied by gravity on opening the bottom or side, as the case may be.

After loading, a tow of three or four dumpers is taken by a tug on the first of the ebb tide through The Narrows to the dumping point, designated by the Supervisor of the Harbor. Each scow carries a crew of two men, paid together $9 50 per day, and each tow has a Government Inspector who is taken aboard from a patrol-boat at The Narrows. At this point, the condition of the pockets—whether full or otherwise—is inspected. From The Narrows the tow proceeds to the dumping ground, which, for garbage or mixed garbage and ashes, during the summer, is 20 nautical miles east-southeast* (magnetic) of Scotland Light-Vessel, 22 miles east of Long Branch, N. J., 13½ miles south of Short Beach, Long Island, and about 36½ nautical miles, or 42 statute miles, from the Battery. In winter, dumping is permitted at a point 12 nautical miles east-southeast of Scotland Light-Vessel, and ashes may be dumped 4 miles beyond the light-vessel. At a point about 3 nautical miles beyond Scotland Light the tow is inspected by a Federal patrol-boat both on the outbound and inbound trip, whose reports, compared with those of the inspection at The Narrows indicate whether surreptitious dumping has taken place in the interval. As an additional check the passage of tows is reported by the Captain of the light-vessel. The tow then proceeds to the dumping point designated by the Inspector, stops, dumps the cargo of each dumper, and returns to port. In favorable weather, that is, when not too calm, it takes about 15 min. to empty a loaded dumper; on the other hand, in a quiet sea some manual assistance is required. During 1924, 494 tows averaging 2.8 dumpers per tow went to the dumping ground, the round trip, probably 90 or 100 miles, taking ordinarily about 22 hours and costing, including Government inspection, $190 per dumper.

As the average load per dumper is $\frac{1\ 728\ 774}{494 \times 2.8} = 1\ 250$ cu. yd., the cost for disposal is $\frac{190}{1\ 250} = 15$ cents per cu. yd.

* By a recent order this has been changed to south-east by east, making the dumping point 21 miles east of Long Branch and 16¾ miles south of Short Beach.

From Table 10 the cost of dumping ashes at sea is seen to be $0.041 per cu. yd. and that of garbage, $0.147 per cu. yd. after loading at the water-front dump.

This is probably the cheapest method of disposing of the garbage of a seaport like New York, but it is open to objection as a sanitary measure and has always been considered a temporary makeshift.

TABLE 10.—MATERIAL DUMPED AT SEA IN 1924.

From:	ASHES.		GARBAGE.	
	Cubic yards.	Cost.	Cubic yards.	Cost.
Manhattan and The Bronx.....	1 051 774*	$42 845	1 138 513	$195 405
Brooklyn.....	.....	.....	590 261	59 300
Total.....	1 051 774	$42 845	1 728 774	$254 705

* Including 439 cu. yd. of garbage.

A considerable part of the material sinks and is no doubt rapidly disposed of by fish and other marine life, but the floating residue, is unsightly and, with winds within a quadrant of southeast, there is always the possibility of its reaching the shore and defiling the beaches. The Metropolitan Sewerage Commission stated that the rate of travel toward the shore "when the tide was favorable and the wind blowing landward at a rate of about 5 miles per hour" was more than ½ mile per hour, and an inspection of the beaches in 1906 showed evidence of garbage more than 50 miles from the dumping point, most of which was "unquestionably" from this source. The dumping point is now farther from shore, but there is complaint of occasional pollution of bathing beaches from drifting garbage the source of which will be the subject of special study during 1926.

The City recognizes the inadvisability of continuing this method of disposal and is extending its policy of incineration so that within a few years the elimination of dumping at sea may be expected. A serious difficulty in most cities, but one acutely encountered in Manhattan, is the strong and effective opposition of local interests to establishing a garbage plant in the neighborhood—a matter that has already caused much delay; but with the abandonment of reduction at the Barren Island and Staten Island plants and the opposition to dumping at sea, the City may be said to be fairly committed to the policy of incineration for the disposal of its garbage.

Disposal by Incineration.—In the entire city there are now twelve incinerators handling garbage and rubbish; sixteen handling rubbish only,* and three handling all refuse. Omitting one of the last, which is experimental in character, those handling garbage are of the types noted in Table 11.

Others are in contemplation in most of the Boroughs, but in Manhattan, especially, great difficulty has been experienced in obtaining sites acceptable

* All in Brooklyn.

to property owners. As new incinerators are brought into operation the quantity of garbage dumped at sea will be reduced until this method of disposal can be entirely abandoned. According to Alfred A. Taylor, Commissioner of Street Cleaning, if funds and sites are made available this may be accomplished in about four years.

TABLE 11.—TYPES OF INCINERATORS USED.

	NUMBER AND NOMINAL CAPACITY, IN TONS PER DAY.				
	Decarie.	Sterling.	Heenan.	Heenan-Froude.	Total.
For Garbage and Rubbish:					
Manhattan	1 -300	1-320	....	.....	2 - 620
The Bronx	.....	.....	None	.....	.....
Brooklyn	.....	1-100	....	1-100	2 - 200
Queens	1 - 70 4 -100 3*-150	.....	....	.....	8 - 920
For All Refuse:					
Richmond	.....	.....	1-50	1- 50	2 - 100
Total	9*-1220	2-420	1-50	2-150	14*-1810

* One of these under contract.

The incinerators handling rubbish only will take care of 50 cu. yd. per hour, the twelve handling rubbish and garbage have a total nominal capacity of 1 740 tons per day, and the three handling all refuse may be put together at about 150 tons per day. The garbage disposed of in this way in 1924 in Manhattan amounted to 109 297 cu. yd.; in Brooklyn, 56 410 cu. yd.; and in Richmond, 26 437 cu. yd. In the Borough of Queens, 105 633 cu. yd. of straight garbage, 608 949 cu. yd. of rubbish, and 60 610 cu. yd. of garbage and rubbish mixed, were incinerated.

Of the incinerators mentioned, those in Brooklyn are owned and operated under a yearly contract by the Brooklyn Ash Removal Company. Within the past two or three years, two 100-ton incinerators for burning garbage and rubbish have been placed in operation, while sixteen furnaces of about 100 cu. yd. capacity per day for rubbish only, are situated about the Borough. Each of these plants disposes of what remains after picking over 1 000 to 1 500 cu. yd. of rubbish per day.

Each unit is hand-stoked and has three grates of a total area of 72 sq. ft., with corresponding ash-pits. Ashes are used for filling land at Corona, in the Borough of Queens. It has been difficult to obtain such complete separation that the garbage and other putrescible and combustible material left in the ashes will not produce a nuisance in the vicinity of the dump. Mr. R. G. Collins, Jr., Chief Engineer of the Brooklyn Ash Removal Company, has therefore devised a plan which may be generally adopted by his Company and which is now in operation at a plant at Sutter Avenue and Junius Street, Brooklyn. This is the experimental plant to which reference has been made.

All refuse except straight steam ashes is first dumped into a pit from the pick-up trucks. From this pit, it is fed to a moving belt from which paper, rags, boxboard, and metal are salvaged and the metal removed by a magnetic separator, the residue being fed from the belt to an Illinois stoker chain grate traveling at a rate of from 30 to 120 ft. per hour. The grate surface is 6 by 30 ft. To obtain uniform incineration the material is kept stirred and evenly distributed by two "agitators" consisting of water-cooled pipes rotating slowly in a direction opposite to that of the grate, as spokes on a transverse horizontal axis. A forced draft applied through tuyères spaced 2 ft. apart under the grate insures complete incineration without auxiliary fuel. The residue is received at the end of the grate and removed in a lateral direction by a drag scraper delivering to a car or truck. As observed, this residue was fine ash without clinker, and contained some cans and small metals that escaped removal. Furnace gases pass through an old furnace improvised as a combustion chamber and thence to a 100-ft. steel stack screened at the top. This plant takes about 50 cu. yd. per hour, the weight per cubic yard varying from 300 to 400 lb. in summer to 900 to 1 200 lb. in winter.

Advantages claimed for this type of incinerator are: (1) The ability to destroy the putrescible and combustible material received with the poorly separated pick-up collections, thus avoiding a nuisance at and near the dump; and, consequently (2), the necessity for separation; and (3) complete incineration with salvage of valuable material and a minimum of manual labor.

For much of the information given, the writer is indebted to Messrs Alfred A. Taylor, Commissioner of Street Cleaning, and E. P. Greene, Superintendent of Final Disposition, City of New York; Carl Gerks, Superintendent of Street Cleaning, Borough of Queens; Theodore S. Oxholm, M. Am. Soc. C. E., Consulting Engineer, Borough of Richmond; and R. G. Collins, Jr., Chief Engineer, Brooklyn Ash Removal Company.

Addendum.—The quantity of refuse handled in 1924 in the Boroughs of Manhattan, The Bronx, and Brooklyn is shown in Table 12.

TABLE 12.—Refuse Collected, in Cubic Yards.*

From:	Ashes.	Garbage.	Rubbish.
Manhattan	2 944 359¾	930 246½	2 496 361½
The Bronx	690 259	317 488	519 862
Brooklyn	2 173 151¾	590 261½	2 461 936¾
Total	5 807 770½	1 837 991	5 478 160¼

* The number of tons cannot be stated, but weights per cubic yard that have been stated or assumed by various authorities, although varying somewhat, lead to the conclusion that both ashes and garbage in these boroughs average about 1 100 lb. per cu. yd. and that rubbish, although subject to a greater variation, may be fairly assumed at about 150 lb. per cu. yd. The tonnage for 1924 would therefore amount to 4 616 030, or 12 630 tons per day from these boroughs.

In 1924, the refuse from Manhattan and The Bronx was disposed of as follows:

1 138 951½ cu. yd. of garbage was dumped at sea, 19* nautical miles in summer and 12 nautical miles in winter, east-southeast of Scotland Light-Vessel;

1 051 335 cu. yd. of ashes were dumped 12 nautical miles beyond Scotland Light-Vessel; and,

3 949 545 cu. yd. of mixed ashes and rubbish were disposed of by dumping on Riker's Island.

The quantities so disposed of each month are given in Table 13.

TABLE 13.—Amount, in Cubic Yards, and Disposition of Refuse from Manhattan and The Bronx in Each Month, 1924.

Month.	To Sea.		To Riker's Island.	To 56th Street Incinerator.	
	Garbage.	Ashes.	Ashes and rubbish.	Garbage.	Rubbish.
January	80 689	109 124	423 546	.....	392
February	65 306	98 878½	410 802	.....	6 818
March	66 268	109 330½	415 914¼	2 632	31 714
April	80 125	84 550	381 537¾	2 130	38 206
May	104 089	62 352	343 751¾	12 407	39 109
June	117 677	29 570	340 304	13 614	39 056
July	120 952	35 838	375 169¼	13 492	49 600
August	119 434	47 420	294 631¾	13 042	52 020
September	119 630½	65 631½	202 040¼	12 768	50 758
October	120 155	83 470¾	249 099½	12 624	52 728
November	81 488	127 533½	244 842	12 288	29 438
December	63 174	197 636½	267 906¼	11 300	57 546
Total	1 138 951½	1 051 335¼	3 949 544¾	109 297	467 385

In Brooklyn, 590 261 cu. yd., or 324 644 tons, of garbage, including some ashes, and 2 173 151 cu. yd., or 1 195 233 tons, of ashes, were dumped at sea; and 2 461 936 cu. yd., or 135 307 tons, of rubbish were disposed of by incineration.

The amounts for each month are given in Table 14.

TABLE 14.—Disposition of Refuse from Brooklyn for Each Month, 1924, in Cubic Yards.

Month.	To Sea.	To Land Dumps.	To Incinerator.
	Garbage.	Ashes.	Rubbish.
January	35 697½	250 474	185 024
February	31 944	231 734	157 470
March	37 254½	250 218½	182 000½
April	41 855½	204 344	204 194
May	52 779	161 794½	229 157
June	58 552½	130 779	222 080½
July	67 590	126 763½	238 083½
August	61 529½	111 242½	206 056½
September	64 313	112 068	207 371½
October	55 629	149 779	225 304
November	42 551½	198 413	197 505½
December	40 565	250 541	207 739
Total	590 261	2 173 151	2 461 936

* This was increased to 20 nautical miles in 1925.

In the Borough of Queens 1 563 000 cu. yd. of refuse were collected in 1924, of which 780 257 cu. yd. were disposed of on inland dumps and 8 250 cu. yd. in grading streets, while 775 193 cu. yd. were incinerated.

In Richmond, all refuse is destroyed in incinerators or disposed of without separation on land dumps. Of the 141 391 cu. yd. collected in 1924, 26 437 cu. yd., or 12 781 tons, were incinerated and 114 954 cu. yd. disposed of on four land dumps. In addition to the 12 781 tons of household refuse, there were 5 941 tons of trade wastes, making a total of 18 722 tons of refuse handled by incineration—an average of 51 tons per day. Table 15 shows the quantity collected and disposed of in 1924.

TABLE 15.

Month.	Collected, in cubic yards.	Household Wastes.			Trade Wastes.	Total.
		To Land Dumps.	Incinerated.		Incinerated.	Incinerated.
		Cubic yards.	Cubic yards	Tons.	Tons.	Tons.
January	12 841	10 677	2 164	1189.3	565	1 754
February	11 265	9 429	1 836	1067.7	489	1 557
March	12 106	10 262	1 844	1107.8	503	1 611
April	11 235	9 668	1 567	880.0	430	1 310
May	15 033	13 023	2 010	961.7	493	1 455
June	10 289	8 144	2 145	1072.7	391	1 464
July	10 871	8 681	2 190	989.8	450	1 440
August	10 491	7 718	2 773	1059.2	567	1 626
September	10 036	7 060	2 976	1132.4	499	1 631
October	10 792	8 556	2 236	1133.1	327	1 460
November	11 516	9 026	2 490	1091.2	579	1 670
December	14 916	12 710	2 206	1096.1	648	1 744
Year	141 391	114 954	26 437	12 781.0	5941	18 722

The average weight of mixed household refuse per cubic yard varied from 760 lb., in September, to 1 200 lb., in March, with a yearly average of 966 lb.

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION

Discussion*

By Frederic H. Fay, M. Am. Soc. C. E.

Frederic H. Fay,† M. Am. Soc. C. E. (by letter).‡—The author has ably described the present condition of the New York State Barge Canal and the apparent reasons why at this time it is used to such a small part of its capacity. Further, he has outlined improvements both in canal structure and in canal operation which should be carried out to make the Barge Canal of the greatest possible usefulness. Because of unfortunate conditions in the past, the canal is even now receiving much undeserved and unfavorable criticism.

In the past, even from its beginning in 1825, the Erie Canal has been of inestimable value in building up New York State and the great Middle West. It was a leading factor in making New York City the first port of the Nation and also in helping it to retain this position.

During the latter part of the Nineteenth Century the railroads were expanding at a rapid rate and reaching out for traffic in every direction. Gradually the greater part of the commerce of the canal was absorbed by the railroads which have retained their grasp on this commerce to the present day. The time has now arrived when the economic needs of the country demand that greater use be made of water routes to the relief of existing rail lines. This is best appreciated by a comparison of the present-day railroad traffic with that of a generation ago. There has been a remarkable increase in traffic on the railroads during the last thirty or forty years with no comparable increase in railroad mileage. In fact, the last few years have witnessed an actual decrease in miles of road operated. In 1890 the freight ton-mileage on all the railroads in the United States was 76 000 000 000 ton-miles; in 1920, this freight movement was 414 000 000 000 ton-miles, an increase of 443% in 30 years. During this period, the total miles of road had increased only 66% (from 156 404 to 259 941) and the total length of all main-line tracks had increased only 79% (from 166 164 to 296 835 miles). Freight movement, therefore, increased in this 30-year period nearly seven times as fast as the increase in railroad mileage, and five and one-half times as fast as the increase in main-line track capacity.

* This discussion (of the paper by Roy G. Finch, M. Am. Soc. C. E., presented at the meeting of the Waterways Division, New York, N. Y., January 21, 1926, and published in October, 1926, *Proceedings*), is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, January 21, 1926.

There is a limit to the freight which can be handled economically by the present railroads even if terminal facilities are enlarged. The time is not far distant when no readily conceivable expansion of the present railroad system will suffice to handle the traffic, and greater use must be made of the economical water routes over which many classes of freight may be transported at costs which are only a fraction of those incurred for movement by rail.

The people of New York State, realizing that its canal system had not kept pace with the growing needs of the country, authorized in 1903 the construction of the new canal, now known as the Barge Canal. Some years later they authorized also the building of public canal terminals at important cities and towns along the route of the canal.

The Barge Canal was completed in 1918 when the United States was at war and conditions throughout the whole country were abnormal. The Federal Government took over the operation of the canal and did not relinquish it until 1921. During the period of its construction there was no incentive for the old operators on the Erie Canal to provide modern equipment; they did not know whether the canal would be efficiently managed or that the full depth required would be maintained. On no other canal in this country were operating conditions the same and only by the fullest experiments could the type of boat that would be economical and efficient on the new waterway be determined. It is a long step from the old horse-drawn barge of 300 tons capacity to the modern Diesel electric unit of 1 500 tons capacity. It takes time not only to determine upon and construct this modern equipment, but also to train a personnel for its operation.

All this is being done on the present Barge Canal without the assistance of either the Federal Government or the State. An instance of the time required and the trials necessary for the development of a new waterway route are well illustrated by the barge-line operation of the Mississippi-Warrior Service, under the control of the Federal Government. This Service, after a number of years of Federal operation, is only now "getting on its feet". It cannot be expected that the New York Barge Canal operators, unaided by Government support, can develop the commerce of that canal any faster than the Federal Government can show results from its own experiments.

There are two reasons why a greatly increased use of the Barge Canal is to be expected in the near future. First, the general economic proposition that waterways are now needed for the relief of the rapidly increasing traffic burden on railroads; and, second, the situation that will result from the opening of the new Welland Ship Canal between Lake Erie and Lake Ontario.

The experience in these few early years of Barge Canal operation has demonstrated that types of canal carriers can be provided to transport many classes of freight, particularly bulk commodities, at a cost substantially below that of transportation by rail. Furthermore, if the canal is deepened to 14 ft. throughout its reaches and its canalized river and lake sections, so as to permit the passage of boats built to the full dimensions and full draft permitted by the present locks, the economies of canal transportation will be further increased. With definite assurance that the State of New York will at all times maintain

the waterway at its full usefulness, canal operators will be justified in providing the modern fleets necessary for economical operation, and the canal will then unquestionably carry a greatly increased traffic, some of which is carried at present by rail.

With the opening of the new Welland Ship Canal from Lake Erie to Lake Ontario, the Barge Canal route from Oswego, N. Y., to the Hudson River will become a far more important link in the National transportation system than is now generally realized. At present, there are transported annually on the four Upper Great Lakes, in modern large lake steamers, from 100 000 000 to 125 000 000 tons of freight at a transportation cost of one-seventh to one-tenth the cost per ton-mile of the transportation of similar commodities by rail lines. The Great Lakes fleet of vessels, with a normal loaded draft of 20 ft. or more, cannot go east of Lake Erie owing to the limitation of depth, 14 ft., of the present Welland Canal.

The Dominion of Canada is expending approximately $115 000 000 in the construction of the new ship canal to replace the present Welland Canal in order to permit the large bulk freight carriers of the four Upper Great Lakes to come to the eastern end of Lake Ontario. This will increase by nearly 200 miles the length of cheaper water haul in these vessels, and decrease correspondingly the distance over which freight must be carried by small vessels or by rail, at higher freight rates, to the seaboard.

The reason for the Canadian Government's heavy investment in the New Welland Ship Canal is shown by the following official statement:

"It is expected that upon the completion of the New Welland there will be a reduction of about 2 cents a bushel in the freight rate on east-bound grain shipments, and that a large proportion of Canadian grain now being shipped by Buffalo and New York will be diverted to the St. Lawrence route".*

In other words, through the New Welland Ship Canal, Canada expects to divert to the Port of Montreal much of the export grain trade now handled at the Port of New York. The rapid rise of Montreal as a port of export for grain, as contrasted with New York, is shown by Table 1.

TABLE 1.—GRAIN EXPORTS FROM NEW YORK, N. Y., AND MONTREAL, QUE., CANADA, IN BUSHELS.

Year.	New York.	Montreal.
1924	105 738 000	152 069 655
1923	97 081 000	118 614 315
1922	127 438 000	148 667 983
1921	68 743 000	126 539 459
1920	83 101 000	57 897 357
1919	90 640 000	46 507 299
1918	63 391 000	62 484 000
1917	119 590 000	60 065 000
1916	153 961 000	70 688 000
1915	126 614 000	44 583 000
1914	61 844 000	75 891 000
1913	65 496 000	46 716 000
1912	53 523 000	38 811 000
1911	33 090 000	30 964 000
1910	17 898 000	27 968 000
1909	22 894 000	27 409 000

* Canada Year Book, issued by Dominion Bureau of Statistics, 1922–23, p. 657.

Prior to the World War, the Ports of New York and Montreal were about even in the quantity of grain exported. During the war years of 1915, 1916, and 1917, grain exports from New York were unusually heavy and much in excess of those from Montreal. Beginning with 1921, Montreal has exported each year a quantity substantially in excess of that exported from New York, the averages for the four years, 1921 to 1924, inclusive, being for Montreal, 135 223 000 bushels, and for New York, 99 738 000 bushels. In 1924, Montreal exported more than 152 000 000 bushels as against less than 106 000 000 bushels from New York, Montreal's excess over New York that year being about 44 per cent. Thus, Montreal has already surpassed New York as a port of grain export, and Canada expects through the New Welland Ship Canal to divert still more grain from New York to the leading port of the Dominion.

There are two factors that will enable New York to retain its share of the export grain trade. The first is the fact that under existing treaty regulations the Welland Ship Canal, like all other waterways of the Great Lakes, will be free alike to vessels of United States and Canadian registry. The second factor is that through the development of Oswego as a port for the accommodation of modern lake vessels, and the utilization of the Barge Canal route from Oswego to the Hudson River, the same savings on the transportation of ex-lake grain to New York may be effected as on transportation to Montreal. Thus, upon the opening of the Welland Ship Canal in 1930, the Barge Canal, coupled with the development of the Port of Oswego, will be of outstanding importance in safeguarding the export grain trade of the Port of New York, which is vitally needed to supply the bulk cargoes required for outbound vessels. Through the agency of the Barge Canal, New York will be able to maintain its present parity with Montreal as a port of grain shipment.

In connection with the need for development of the Port of Oswego by the Federal Government an extended investigation has been made of the commerce of the Great Lakes region and of the future traffic of the Barge Canal System. The conclusions reached are, briefly, that the Barge Canal route between Oswego and the Hudson River will probably carry, shortly after 1930, a traffic well in excess of 2 000 000 tons, or approximately equal to that of the entire Barge Canal System to-day. Furthermore, Barge Canal traffic to and from Buffalo will apparently be maintained at approximately its present tonnage figures, although there will be a decrease in the quantity of grain moved from Buffalo which should be offset by the transportation of other commodities. Shortly after 1930, therefore, it is to be expected that the total traffic of the Barge Canal System will be double the present traffic.

Not only will the importance of the Barge Canal to the Port of New York be greatly enhanced, but it will be a vitally important factor also in providing economical movement of freight between the new seaport at Albany and the Great Lakes region.

The people of the State of New York have builded wiser than they knew in providing the modern Barge Canal. It is only at the beginning of its career of useful service, not only to New York State but to the whole nation. A ship canal from the Great Lakes to the sea is of doubtful economic value and in any case cannot be secured for many years. As Mr. Finch has suggested, the deepening and widening of the present Barge Canal may well afford the economical outlet to the sea which the people of the Middle West are now demanding.

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMN RESEARCH

Discussion*

By E. H. Salmon, Esq.

E. H. Salmon,† Esq. (by letter).‡—The Special Committee on Steel Column Research is to be congratulated on its Progress Report,§ and especially on the lines which it has laid down for its investigations. American research has done a great deal to extend the sum total of knowledge on this important subject, and the completed results of the Committee's efforts will be welcomed by engineers of all nationalities.

In particular, it is most interesting to observe that under Part D the Committee proposes to attack the question of the behavior of the column as part of a structure. Careful study of the problem long ago convinced the writer that the next great step forward in the column theory will come from a realization that the behavior of a column forming part of a structure is quite different from that of a textbook example or a laboratory specimen, and he believes that investigation along these lines will lead to conceptions regarding the column in practice of a very different nature from those commonly accepted.

Except, perhaps, for perfectly new materials, the day for long series of laboratory experiments on specimens of different lengths is over. Laboratory experiments will be necessary, but they will be directed to elucidate quite other questions than the mere effect of increasing length, which, it is now generally recognized, has not the great influence on ordinary short practical columns that the early engineers, misled by Euler's theory, supposed.

It is most unfortunate that the Euler theory, perfectly correct though it be, has so obsessed the minds not only of all the early investigators on this subject, but of most of the later European workers, as too often to obscure the fact that the theory is quite inapplicable to the ordinary practical column, and that conclusions drawn from the theory do not hold for such members. Both theoretically, and experimentally, effort has been directed to prove or disprove the Euler theory, rather than to investigate what happens to the practical column. Even in America, far freer from the influence of tradition,

* This discussion (of the Progress Report of the Special Committee on Steel Column Research, presented at the Annual Meeting, New York, N. Y., January 20, 1926, and published in March, 1926, *Proceedings*) was received too late to be published with the Report in *Transactions*, Vol. 89 (1926), but is published in *Proceedings* in order that the views expressed may be brought before all members.

† (Clark & Standfield), London, England.

‡ Received by the Secretary, July 23, 1926.

§ *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, p. 146.

the utilization of flat and pin ends in laboratory specimens has resulted in a mass of experimental data that is not at all closely related to the practical column, or as the Committee expresses it, "can hardly be applied to columns in structures".

If the Committee will pursue its investigations under Part D to their logical conclusion, the writer firmly believes that much of the old column theory will have to be replaced by a new theory with frame distortion as its underlying basis. The old ideas regarding the deflection curves of columns will be found to be quite beside the mark; such evidence as there is points to **S**-shaped bending as the general rule, and it is not improbable that secondary stresses due to frame distortion will be found to be of greater importance than the bending set up by column defects.

Here a word of criticism seems desirable. A mere theoretical investigation will be quite inadequate. What is wanted is experimental observation of the behavior in actual structures—in bridges, roofs, and buildings. The first necessity is to find out what really happens. The difficulties are admitted, but there will be no real progress until they are faced and overcome. In particular the "give" of the riveted end connections, which will have a great influence, should be very carefully investigated. In this connection the recent German proposal,* to determine the degree of end fixity by observations on the dynamic vibrations of the member, might be worth the Committee's consideration.

Turning to one or two details in the report, the writer was much interested to observe that the points of inflection, or rather of no bending moment, had been experimentally located in certain specimens at $0.56\,l$ apart.† He has shown‡ that if a column, perfectly "direction-fixed" at the ends, has initial curvature, the Eulerian conclusion, that points of inflection lie $0.5\,l$ apart, no longer holds; in fact, the position of the points of no bending moment vary with the magnitude of the load.§ At very small loads the "free length", $q\,l$, equals $0.578\,l$, corresponding to a direction-fixed beam. As the load increases, the value of q drops slowly to 0.561, when the load is one-quarter of Euler's crippling load, and to the Eulerian value of 0.5 at the failure point. Since all members have initial curvature, the experimental value of 0.56, which agrees exactly with the practical value at the one-quarter Euler load, may be regarded as evidence that the degree of end fixing in the experiments was very good, and incidentally that it would have been more correct if the Committee had used the ordinary value, $\frac{4\,\pi^2\,E}{\left(\frac{l}{r}\right)^2}$, for the Eulerian crippling load when q has dropped to 0.5. An interesting corollary to the foregoing is that in symmetrical cross-sections, at small loads, the stress due to bending at the ends of a perfectly direction-fixed column with initial curvature is double that at the center.‖

* "Bauingenieur," W. Engler, Vol. 7, pp. 343–348.

† *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, p. 191.

‡ "Columns," p. 71, and Table, p. 72.

§ *Loc. cit.*, Fig. 51, p. 173.

‖ *Loc. cit.*, p. 52 and Fig. 15.

The writer is pleased to observe that the Committee has plotted Johnson's parabola as a standard of comparison. If an empirical formula is to be used, the Johnson parabola has many points in its favor, as indeed Ostenfeld* showed many years ago. One of its advantages, and not the least, is the fact that it lends itself so readily to the use of Asimont's device for the direct determination of the area of the column.†

Although, as the Committee has shown, the initial deflection or eccentricity as determined from the observed bending stresses in carefully prepared laboratory specimens is smaller than the estimates commonly made, it should be emphasized that what is required is the initial curvature likely in practice, and this will certainly exceed that likely in the very carefully prepared specimens of the later American tests.

The writer's estimate, $\frac{l}{750}$, for the probable initial curvature, which the Committee quotes, was obtained by plotting values from a large number of laboratory specimens, mostly carefully straightened. Certainly, the values likely in practice will not be less than this. In addition to this measured want of straightness, any increase due to variation in the modulus of elasticity and other imperfections must be included. In view of the admitted uncertainties of the case, a fairly liberal estimate is to be desired; in fact, some writers have proposed a special factor of safety to be applied to experimental eccentricities. It is well to insist on this distinction, as otherwise the Committee's remarks, although carefully guarded, may well be used as authority for a most inadequate estimate of the probable initial curvature in practice.

The proposal‡ to measure the initial deflection in actual fabricated members would certainly be a step in the right direction, and it is hoped that it will be carried out.

In practical structures, where the ends of the columns are more or less direction-fixed, the eccentricity of loading consequent on bad centering in the testing machine, cannot occur; in fact, a positive eccentricity is incompatible with the idea of fixity in direction, which implies that the eccentricity is negative. Initial bending moments, which may quite overshadow the effects of eccentricity, positive or negative, are, however, always possible in framed structures.

It is hoped that the Committee will continue its investigations on the subject of secondary flexure. There is no doubt that in built-up members it is the strength of the individual part which determines the strength of the whole. If secondary flexure is possible the stresses induced by local bending add to the stresses due to primary flexure, and from this point of view real "integral" action is impossible in any but a solid member. It is a far wider question than a determination of the local slenderness ratio necessary in order that the elementary column should reach the yield-point stress before buckling. It is

* "Exzentrische und zentrische Knickfestigkeit," *Zeitschrift*, Verein deutscher Ingenieure, Berlin, December 31, 1898, p. 1462.

† "Eine Knickungs-Formel," *Zeitschrift*, Bayerischer Architecten und Ingenieure Verein, München, Bd. VIII, Heft 6, p. 119; also, "Columns," p. 247.

‡ *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, p. 197.

the combination of local buckling with primary flexure that is important. This combination can be traced in all experiments in which secondary flexure is possible, from Hodgkinson's early experiments on thin tubes (1849)* onward. The writer made a careful study of the 1882-85 Watertown Arsenal experiments on built-up wrought-iron columns; it is surprising how superior the lattice-braced channel columns appear compared with the "solid" sections built up of plates and angles.† This is due, he believes, to the greater resistance to local flexure in the former, in which $\frac{l}{r}$ for local buckling was much less than in the latter. It is more than probable that local flexure plays a far greater rôle in the strength of all parts in compression than is generally recognized.

Any one who has made observations with strain-gauges on box-like structures cannot but have been struck with the influence exerted by local flexure; indeed, the writer has become very skeptical regarding all such measurements on thin material unless strain-gauges have been used on both sides of the plates.

The consequences resulting from the combination of primary and secondary flexure are still very imperfectly recognized. For example, as long ago as 1908, Krohn‡ pointed out that it necessitated a greater moment of inertia of cross-section for bending in the direction influenced by the local flexure than in a direction not thus influenced. It is a commonplace in nearly every textbook and handbook, however, that I should be the same about each principal axis; and formulas, applicable only to solid sections, are every day applied to built-up members in which very considerable local flexure can occur. An exhaustive examination of this question, with the prestige of the Committee behind it, might do much to put the matter on a more reasonable basis.

The question is connected indirectly with that of shear stress in columns. This, when not influenced by frame distortion, is known to be relatively small, which appears to the writer to be some justification for the more common use of batten-plate columns than is perhaps usual. Batten-plate columns are cheaper, easier to construct and to paint than a lattice-braced member. Experiments by Emperger§ and others go to show that, if properly designed, they are as efficient as any other column type. (To design them by the solid-column formula is, of course, absurd.) This is again a point on which an authoritative pronouncement by the Committee would be of great service to the profession.

* "The Britannia and Conway Tubular Bridges," by Edwin Clark, Lond., 1850.

† "Columns," Fig. 54, p. 183.

‡ "Beitrag zur Untersuchung der Knickfestigkeit gegleiteter Stäbe", *Zentralblatt der Bauverwaltung*, Berlin, October 21, 1908, p. 559.

§ "Welchen Querband bedarf eine Eisensäule," *Beton und Eisen*, Wien, February 19, 1908, p. 71, etc.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ANDREW OSWALD CUNNINGHAM, M. Am. Soc. C. E.*

DIED MAY 9, 1926.

Andrew Oswald Cunningham was born in Rangoon, British Burmah, on July 8, 1866, the son of Maj. Gen. Percy S. Cunningham, of the British Army, and Anna Sarah (Stroud) Cunningham. His father held many commissions of trust in the Government Service while in Burmah.

Mr. Cunningham attended South-Eastern College, at St. Lawrence, England, from 1879 to 1883, when he came to the United States. He took up railroad work and was, successively, Rodman and Leveler with the Northern Pacific Railroad Company from 1886 to 1888. He then engaged in land surveying in North Dakota for two years.

In 1890, at the age of 24, he entered the University of Minnesota and was graduated in 1894, with the degree of Bachelor in Civil Engineering. His classmates recall with affection his likeable personality and his sense of humor. Having attended school in England, Mr. Cunningham was an expert at Rugby football and astonished coaches and students alike with his proficiency. He took his university work more seriously than the average student because of his more mature years, and found less time for the lighter phases of student life, although he was generally popular. While pursuing his engineering studies he was engaged in constructing insurance maps of the City of Minneapolis.

Following his graduation Mr. Cunningham became associated with the Gillette-Herzog Manufacturing Company, of Minneapolis, in the capacity of Draftsman and, in 1895 and 1896, was Assistant Engineer of the Company. From 1896 to 1898 he was Southern Agent for the Schultz Bridge and Iron Company of Pittsburgh, Pa. Succeeding engineering engagements were with the Pittsburgh Reduction Company, in 1898 and 1899; general construction and engineering, in 1899 and 1900; and in charge of estimates, designs, and bids for the American Bridge Company, Cleveland, Ohio, from 1900 to 1902.

In 1902 Mr. Cunningham removed to St. Louis, Mo., where he became Bridge Engineer of the Wabash Railroad Company. He was promoted to the position of Chief Engineer in 1905 and continued in that capacity until 1923 when he was made Consulting Engineer. He resigned from this position in 1924 to engage in consulting practice. He was Consulting Engineer of the Terminal Railroad Association of St. Louis from 1924 until his death, which occurred at his home in University City, Mo., on May 9, 1926, at the height of his career.

* Memoir prepared by the following Committee: O. F. Harting, Assoc. M. Am. Soc. C. E., *Chairman*, R. H. Howard, and W. E. Rolfe, Members, Am. Soc. C. E.

Mr. Cunningham was a man of active mentality, great breadth of vision, and marked originality. His resourcefulness in rehabilitating and reinforcing obsolete bridges, increasing their capacity to carry modern loading, by unique methods and original designs, thus avoiding the expense of replacement, won for him much high commendation. He perfected many inventions, and many mechanical devices were the result of his thought and study. He was the first to advocate the use of reinforced concrete in the construction of bridge floors, and the Cunningham mattress for river-bank protection has been widely used.

He was much interested in civic affairs and served a two-year term as Alderman of University City, Mo., beginning in April, 1914. He was elected Mayor of University City in April, 1925, but died before the expiration of his term of office. While serving as Alderman he prepared and negotiated a contract securing to University City one of the best and cheapest street-lighting systems in any city of the United States. During his tenure as Mayor many municipal improvements were carried out under his direction, and his experience and judgment in engineering and construction work were invaluable to the Engineering Department of the City.

The death of Mr. Cunningham is a distinct loss to all with whom he came in contact; his friendship and counsel will be missed by all his former associates and he leaves a void in the ranks of the Engineering Profession that will be difficult to fill.

Mr. Cunningham was a member of the American Railway Engineering Association and the Engineers' Club of St. Louis, in both of which he took an active part, having served as Chairman and member of a number of their important committees.

He was married on July 11, 1903, to Mrs. Georgia Townsend Quinn, of St. Louis. In his home he was a perfect host. His charity was boundless, without regard to sect or creed. It is fitting that his memorial be concluded in the words of his devoted wife and helpmate:

"With him honor came first at all times, then love for his family and friends; he cared only for a peaceful and honorable life, trying to bring happiness where he could."

Mr. Cunningham was elected a Member of the American Society of Civil Engineers on February 5, 1902.

CLARENCE THAYER FERNALD, M. Am. Soc. C. E.*

DIED MAY 11, 1926.

Clarence Thayer Fernald was born in Charlestown, Mass., on November 10, 1866. He was the son of Ichabod Nichols and Abigail (Thayer) Fernald. He received his early education in the public schools in Charlestown, and later attended the Lowell Institute Courses of the Massachusetts Institute of Technology, specializing in scientific studies, advanced mathematics, etc.

* Memoir prepared by H. M. Steward, M. Am. Soc. C. E.

In 1884, Mr. Fernald entered the employ of Mr. J. O. Goodwin, an engineer of Medford, Mass., as a Rodman and remained with him for six years, on general engineering and municipal work.

In 1890, he became connected with the Norfolk and Western Railway Company, as Transitman and Chief of Location Party, remaining with that Company until 1891, when he established, with an associate, a general engineering practice in Melrose, Mass. In 1894, he was appointed First Assistant Engineer in the Sewer Department of Melrose, and had charge of the survey and construction of 30 miles of its sewerage system.

Shortly after the Boston Elevated Railway Company began the construction of the Rapid Transit System in Boston, Mass., it secured the services of Mr. Fernald, his appointment being made in February, 1899. From that time, he had an intimate part in the development of the Rapid Transit System. He devoted the greater part of his twenty-seven years of service with the Boston Elevated Railway Company to this work, which consisted of designing and constructing elevated structures, viaducts, tunnels, subways, and, just prior to his death, the Dorchester Rapid Transit Extension over the right of way of the Shawmut Branch of the New York, New Haven and Hartford Railroad. Mr. Fernald was recognized as an authority on all the varied problems pertaining to rapid transit engineering, and the work which he did in connection with the development of the Rapid Transit System of Boston will always remain as a monument to his engineering ability. He was consistently painstaking and accurate in every detail of research, design and construction, and was also energetic in the carrying out of each project.

In the civic affairs of his home city—Melrose—Mr. Fernald took an active interest. He served for four years as Alderman at Large on its first Board of Aldermen, and, later, for seven years, as Chairman of the Board of Park Commissioners. In addition, he also served on many committees on various civic improvements.

In the Presidential Campaign of 1905, he was a member of the Massachusetts Electoral College, having been elected Presidential Elector from the Eighth Congressional District of Massachusetts, of which District he was for eight years a member of the Congressional Committee.

Mr. Fernald was active in fraternal orders. He was a Past-Commander of Wyoming Lodge, F. and A. M., of Melrose, and a member of Waverly Royal Arch Chapter, Hugh de Payens Commandery, and Aleppo Temple, A. A. O. M. M. Shrine. In the Melrose Lodge of Elks, he was Past-Secretary. He was also a member of the Boston Society of Civil Engineers, New England Street Railway Club, New England Traffic Club, and American Electric Railway Association.

Mr. Fernald was a man of sterling character, and of genial disposition, although he was positive in his convictions. He always enjoyed a wide circle of friends, through his affiliations with a large number of organizations, to which he freely gave of his time and where his presence and advice will be greatly missed. To those who had the pleasure of being associated with him in his work he endeared himself by his marked ability, loyal friendship, and willingness to offer any assistance he was able to give.

He was twice married and is survived by his widow and a daughter.

Mr. Fernald was elected a Member of the American Society of Civil Engineers on October 7, 1908.

CLARK ROBINSON FICKES, M. Am. Soc. C. E.*

DIED FEBRUARY 8, 1926.

Clark Robinson Fickes was born near Orrstown, Pa., on May 27, 1873. He received his education, both elementary and technical, in Iowa City, Iowa, and was graduated from the State University of Iowa in 1893 with the degree of Bachelor of Science. He received the degree of Civil Engineer in 1906.

During the period from 1894 to 1896, Mr. Fickes was Assistant City Engineer and City Engineer, of Iowa City, and designed and directed the construction of storm and sanitary sewers, and streets.

From 1896 to July, 1897, he was engaged as Assistant to R. M. Cannon, Civil and Mining Engineer, at Colorado Springs and Cripple Creek, Colo., computing mineral surveys and supervising the construction of a large ore-treating plant. At the request of the City Council, he then returned to Iowa City to take charge of the construction of three miles of pipe sewers.

In 1898, Mr. Fickes entered the employ of the Chicago, Burlington and Quincy Railroad Company, as Assistant Engineer, in which capacity he was engaged in general railroad maintenance until 1904. During this time he had direct charge of the construction of a large freight yard near Hawthorne, Ill., consisting of 75 miles of track, bridges, and other accessories; made all the preliminary surveys and was responsible for the double-tracking of the main line between Monmouth, Ill., and Burlington, Iowa, which included heavy work, due to grade and curve eliminations. He also had charge of the office work in connection with the double-tracking of the main line through Iowa and designed all minor structures in connection therewith.

From November, 1904, to March, 1905, Mr. Fickes served as Drainage Engineer for the Des Moines County, Iowa, Commissioners, making a survey and an estimate for the drainage of about 50 000 acres of swamp land.

During 1905 and 1906, he was employed as Resident Engineer for the St. Louis Southwestern Railway Company on the construction of seven bridges over the Red River, at Shreveport, La. This was considered a very troublesome and rather difficult piece of work on account of high-water conditions. Concrete caissons were carried to rock, by open dredging, through 85 to 90 ft. of clay and sand.

In 1906, Mr. Fickes again entered the employ of the Chicago, Burlington and Quincy Railroad Company as Assistant to the Bridge Engineer, in charge of maintenance and construction field work on all bridges on the System.

In 1912, the project of extending the Southern Illinois Line of the Burlington System across the Ohio River to a connection with the Nashville, Chatta-

* Memoir prepared by Harrison C. Shafferman, Esq., Miami, Fla.

nooga, and St. Louis Railroad, at Paducah, Ky., was given its first impetus, and Mr. Fickes was sent to Metropolis, Ill., to lay down the bridge line, make the necessary surveys, soundings, etc. During the progress of this work, and on the death of C. H. Cartlidge, M. Am. Soc. C. E., Bridge Engineer of the Chicago, Burlington, and Quincy Railroad Company, and Chief Engineer of the Paducah and Illinois Railroad Company, Mr. Fickes was appointed Chief Engineer and handled successfully all the duties in connection with the position. In the summer of 1917, when the steel work was just about completed on the Metropolis Bridge, he was compelled to resign on account of serious illness contracted during the sinking of the caissons under high pressure.

After regaining his health in the spring of 1918, Mr. Fickes accepted a position with the Valuation Department of the Interstate Commerce Commission, with headquarters at Chattanooga, Tenn.

In June, 1918, he was commissioned a Major of Engineers, U. S. Army, and ordered to France. During the remainder of the World War and until the spring of 1919, he was engaged on the construction of railroad and dock facilities at the proposed American port at Talmont, near the mouth of the Gironde River, where he was Commanding Officer of a camp of 4 000 men, including several companies of the 18th and 32d Engineers, labor battalions, and German prisoners.

On his return to the United States in the spring of 1919, Major Fickes was employed as Assistant to H. N. Rodenbaugh, M. Am. Soc. C. E., who was then Engineer of the Southern District for the United States Railroad Administration, with an office at Atlanta, Ga. In this capacity, he supervised the reconstruction of the Southern Railway Bridge across the Tennessee River near Chattanooga, Tenn., which work was accomplished in less than five months.

In March, 1920, when the Railroad Administration relinquished active charge of the lines, he was called to Washington, D. C., in connection with the work of liquidation, and in April of the same year, he was appointed Regional Director, with an office in Chicago, Ill. His work was carried out so successfully that his duties under that appointment were completed by October, 1921, after which he was called to Washington and given charge of the settlement of claims of about sixty small lines in various parts of the United States. This task was completed in July, 1922.

In June, 1923, Major Fickes accepted the position of Resident Engineer of the Florida East Coast Railway Company, in charge of the reconstruction of the St. John's River Bridge at Jacksonville, the project including the replacing of the single-track structure with double track of a much heavier and most modern type. Under his efficient direction, this bridge, which consisted of twenty-two girder spans, three fixed truss spans, a machinery span, and a 216-ft. Strauss heel trunnion bascule leaf span, a total length of 2 451 ft., with twenty-six piers and two abutments, all pneumatically founded, was constructed at a cost of $2 500 000. The bridge was built at a time when both rail and river traffic was the greatest in the history of the State of Florida, with practically no delay to either.

He was a man of wonderful personality, greatly beloved by those who knew him, and as an engineer his word was authority on all matters pertaining to his

profession. He will be greatly missed by his fellow members and by his many friends.

Major Fickes was elected a Member of the American Society of Civil Engineers on November 7, 1906.

GEORGE MORRALL GADSDEN, M. Am. Soc. C. E.*

DIED APRIL 14, 1925.

George Morrall Gadsden was a native of Charleston, S. C., having been born there on October 19, 1865. His parents were Maj. Christopher S. and Eliza (Morrall) Gadsden, both of whom were of distinguished South Carolina families.

Mr. Gadsden was educated at the Citadel Military Academy at Charleston, having been graduated from that institution in 1886. He afterward entered the employ of the Pennsylvania Railroad Company as a Civil Engineer, remaining with it, at Pittsburgh, Pa., for two years. He then went to Savannah, Ga., and entered the employ of the Plant System, now the Atlantic Coast Line Railroad Company, of which his father was at that time a Vice-President. Subsequently, he resigned that position and engaged in phosphate mining in Florida, where he remained for two years. Having been compelled to leave Florida on account of his health, he rejoined the Plant System as a Civil Engineer. After serving in this capacity for several years, he was elected Director of Public Works of the City of Savannah, and through his excellent work as the head of that Department during several City Administrations, as well as in his other activities, he became a prominent and influential member of the community.

After severing his connection with the Department of Public Works, Mr. Gadsden organized and became the head of the Gadsden Contracting Company. This firm built the sea wall at Tybee Island, Georgia, the Atlantic Coast Line Railroad Bridge over the Altamaha River, and acted as Contractor for all the bridges, on the celebrated road to Tybee Island, over many miles of marshes. Under Mr. Gadsden's direction, his firm also acted as Contractor for many other important projects in the Southeast.

He was a veteran of the Savannah Volunteer Guards, which was organized in 1802 and is one of the oldest military organizations in the country. He organized Company D and was its first Commander. He was a member for many years of the Oglethorpe Club, the social club of Savannah; of Ancient Landmark Lodge, F. and A. M., and of the Benevolent and Protective Order of Elks. He was also identified with the banking interests of Savannah as a Director of the Exchange Bank.

Mr. Gadsden is survived by his widow, two sisters, and one brother, Philip M. Gadsden. His sisters reside in Charleston and his brother in Philadelphia, Pa. His funeral service was held from Christ Church, Savannah, and was attended by his numerous friends; the City flags were displayed at half-mast in his honor.

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* Memoir prepared by George T. Cann, Esq., Savannah, Ga.

While modest and retiring, George M. Gadsden was a man of acknowledged ability in his profession and of the highest integrity in every phase of his life, a faithful friend, and most popular with all who knew him. Not only his friends but the community in which he lived sustained a distinct loss by his death.

Mr. Gadsden was elected a Member of the American Society of Civil Engineers on May 4, 1898.

WILLIAM GALT RAYMOND, M. Am. Soc. C. E.*

DIED JUNE 17, 1926.

William Galt Raymond was born on March 2, 1859, at Princeton, a little town on the Iowa side of the Mississippi River, about 20 miles above Davenport. Through his father, William Henry Vining Raymond, and his mother, Laura Guernsey (Peet) Raymond, he was descended from families prominent in Connecticut history from Colonial times. One of these ancestors, seven generations back, Capt. Thomas Yale, with his stepfather Theophilus Eaton, came from England to New Haven in 1637. His father's work in education took the family much about Illinois and developed in the lad a variety of interests. His mother's frailty of constitution had an important influence in leaving him to his own devices. Far and wide he roamed over many a countryside, and early earned a reputation for vigor, self-reliance, and fertility of resource. One of his earliest interests was in watching machinery and in conducting mechanical experiments.

After assiduously attending the public schools in various cities of Illinois and Kansas, he went from his home in Leavenworth, Kans., to the University of that State, where for three years from 1877 to 1880, in conformity with his father's wishes, he took a course in arts. Subsequently (1881-82 and 1883-84) he completed the Civil Engineering course at Washington University, St. Louis, Mo., from which he was graduated in 1884 with the degree of Civil Engineer. In 1905, he received the degree of Doctor of Laws from this institution. In 1919, he was honored by the University of Michigan with a Doctorate in Engineering.

Mr. Raymond's practical engineering experience began during his summer vacations and the years that intervened between his periods in college. He was successively engaged upon preliminary and construction surveys for the Kansas Pacific Railroad, now a part of the Union Pacific; and for the Kansas City, Fort Scott, and Gulf Railroad, and the Springfield and Memphis Extension of the latter. For a time he was a Resident Engineer in charge of construction.

After a brief period on Mississippi River improvement work, he was appointed, in 1884, an Instructor in Civil Engineering in the University of California. In 1890, he resigned to take charge of the location, under the late Samuel Rockwell, M. Am. Soc. C. E., Chief Engineer, of a railroad through the Coast Range Mountains between the San Joaquin Valley and San Francisco Bay. Subsequently, he engaged in general private civil engi-

* Memoir prepared by S. M. Woodward, and R. B. Kittredge, Members, Am. Soc. C. E.

neering practice, including land, mining, and water-supply surveys, and worked on numerous surveys and plans for drainage, irrigation, and town sites. For a time he was Town Engineer of Berkeley, Calif., planning and executing a considerable amount of municipal work.

From 1892 to 1904, he served as Professor of Geodesy, Road Engineering, and Topographical Drawing at Rensselaer Polytechnic Institute, Troy, N. Y. During this period he was Chief or Consulting Engineer for the Troy Department of Water Supply, designing and building the greater part of the present water supply system of that city. He also built the Troy-New England Electric Railway, and was Consulting Engineer in many cases involving questions of water power and mill rights.

In 1904, he went to the Iowa State University at Iowa City as Professor of Civil Engineering, and, in 1905, was appointed Dean of the newly created College of Applied Science at the same institution, which position he held until his death. In his later years Dean Raymond gave much time to the legal and economic problems of public utilities and wrote extensively in this field. For five years he was a member of the Special Committee on Valuation of Public Utilities of the Society, during which period his industry, energy, and judgment were of great assistance in preparing its report.

Besides nearly one hundred papers published in technical journals and society proceedings, he was the author of: "Plane Surveying" (1892); "Elements of Railroad Engineering" (1908); "Railroad Field Geometry" (1910); Railroad Field Manual for Civil Engineers (1915); section on "Surveying" of Mark's Mechanical Engineers' Handbook (1916); "What is Fair?" (1917); and "The Public and Its Utilities" (1925).

Dean Raymond was always active in matters of professional engineering organization. He was a member of the Society for the Promotion of Engineering Education from its organization in 1893, having served as President in 1911-12; an active member and Past-President (1913-14) of the Iowa Engineering Society; and a member and Past Chapter President (1915-16) of Sigma Xi. He was also a member of the American Railway Engineering Association; the Rensselaer Society of Engineers; the Chicago Engineers' Club; Phi Kappa Psi; Triangle; and Tau Beta Pi. He was affiliated with the Elks, and with the Masonic order, in which he held high rank, having been Past Master of Iowa City Lodge No. 4; a Shriner; a Knight Templar; a Thirty-second Degree Mason; and a Knight Commander of the Court of Honor.

In his community Dean Raymond was an active citizen and leader, a Director in various corporations, a Director of the Chamber of Commerce, a member of the Rotary Club, a Trustee and, for several years, Sunday School Superintendent of the Presbyterian Church, and, for six years, a member of the School Board. To the duties of the latter position he gave unstintingly of his time throughout an arduous period during which most of the old school buildings of the city were replaced by modern structures.

Ever an ardent lover of music, Dean Raymond was for eight years Manager of an Annual Concert Course in Iowa City, for which he secured the highest professional talent. When the University radio broadcasting station was opened, the responsibility of its operation was assigned to him, and to the

development of its educational and recreational features he gave much personal attention.

At Fort Scott, Kans., on July 1, 1885, Dean Raymond was married to Helen Williams, daughter of Edwin R. Bay, a pioneer druggist, of Chicago, Ill. Of this marriage four children were born: Margaret Leonard, wife of Harry P. Hammond, M. Am. Soc. C. E.; William Yale, Sales Superintendent; Edwin Bay, Banker; and Laurence Guernsey (deceased). For the last few months before his death Dean Raymond had not been in the best of health. In May, 1926, he entered the University Hospital for an operation, after which he appeared for about two weeks to be making excellent progress, when an embolus in the lung quickly brought on the end. Mrs. Raymond and three children survive him.

Perhaps Dean Raymond's deepest influence was exerted through his teaching and through the local affairs of the communities in which he lived. As an educator he was unusually progressive and much in advance of the general ideas of his time. In engineering college education he was early and always an energetic advocate of broad training in fundamentals as contrasted with specialization. He believed enthusiastically in recognizing individual differences among students and, as far as he was able, adopted scholastic methods to permit of individual variations in the rate of doing college work. His intimate association with his students, his sympathy with them, and his devotion to their interests attracted them to him in a remarkable way. In his innumerable contacts with students over matters of discipline and the conduct of college activities, his sterling and outspoken courage and sincerity, combined with his high ideals of accomplishment, wielded an influence that permeated the entire student body. He touched the lives of students and penetrated their aims and ambitions to an extent impossible to measure.

Dean Raymond endeared himself to his associates in a way that passes on his influence to the future in an outstanding degree. Honesty and fairness, conscientious industry, and intellectual vigor were combined in him with an incessantly active nature, a hearty frankness, and a tender sympathy. All these inspired a deep loyalty and affection among his colleagues and students alike. With his departure an outstanding personality is lost, but the beneficent influence of that life and character will be felt and cherished for long years to come.

Dean Raymond was elected a Member of the American Society of Civil Engineers on October 3, 1894.

RICHARD WILLETTE SHERMAN, M. Am. Soc. C. E.*

DIED MAY 23, 1926.

Richard Willette Sherman, the eldest son of Gen. Richard U. and Mary F. Sherman, was born at Utica, N. Y., on December 4, 1848. He attended the public schools of Utica in 1856 and 1857. In the spring of 1858 the family

* Memoir compiled from information on file at the Headquarters of the Society.

moved to a farm in the adjoining Town of New Hartford where he attended the district school and, later, during two winters, Bryant and Stratton's Commercial School, in Utica.

During 1864 and 1865, on account of the absence of his father, who was Assistant Clerk of the House of Representatives at Washington, D. C., he was in charge of the farm.

In 1866, Mr. Sherman secured a position as Rodman in the Engineer Corps of the Utica, Chenango and Susquehanna Valley Railroad Company (now the Delaware, Lackawanna and Western Railroad Company) between Utica and Sherburne, N. Y. In connection with this work and subsequent employment, and with the assistance of his superiors, he studied civil engineering, devoting as much time as possible to this task. In the spring of 1867, he accepted a similar position with the Albany and Susquehanna Railroad Company (now the Delaware and Hudson Railroad Company) between Bainbridge and Binghamton, N. Y., having been attracted by the tunnel work then in progress on that line.

From 1868 to 1872, he held the position of Assistant Engineer for the New York and Oswego Midland Railroad Company (now the Ontario and Western Railroad Company), in charge of surveys and the construction of the road between Oneida and Central Square, N. Y. On the completion of this work, he was appointed Resident Engineer of the Company's line between Walton and East Branch, N. Y., including the Hawk's Mountain Tunnel. During 1870, he was also Assistant Engineer in charge of the construction on the New Jersey Midland Railroad.

In 1873, he accepted the position of Assistant Engineer in charge of construction and surveys of all new lines for the Erie Railroad, but resigned in 1875 to go into the contracting business on the New York and Canada Railroad, near Pittsburgh, Pa. In 1876, Mr. Sherman, in partnership with Dr. Emmett Flagler, constructed the West Troy and Green Island Water-Works, as well as water-works plants at Gloversville, Richfield Springs, and Walton, N. Y. The firm also constructed public works in Vermont and Ohio, and made improvements on the Champlain Canal for New York State.

The firm of Sherman and Flagler was dissolved in the fall of 1879 when Mr. Sherman went to Cuba to build a gas plant to supply the entire city of Havana. He returned to the United States in 1880 to construct by contract the water-works for Greenwich, Conn.

In 1881, Mr. Sherman, in partnership with Mr. Michael McDonough, secured a large contract to construct a number of miles of the West Shore Railroad, near Haverstraw, N. Y. This work was completed in 1882 and the firm of Sherman and McDonough continued an extensive and successful contracting business, constructing water-works at Lansingburgh, Sandy Hill, Little Falls, Camden, Canastota, New Berlin, and Oneonta, N. Y., as well as sewers at Lansingburgh and Amsterdam, N. Y., and South Framingham, Mass., together with canal work under numerous contracts for the State of New York.

From 1886 to 1888, Mr. Sherman served as City Surveyor of Utica. While in this position, and with the co-operation of Mayor T. E. Kinney, the public

works of the city were rapidly brought up to date. The program laid down by Messrs. Sherman and Kinney has been continued, the City of Utica spending on an average about $250 000 per year on pavements, sewers, etc.

In 1888, the firm of Sherman and McDonough was incorporated as the Troy Public Works Company with a capital of $30 000, and continued to carry on a successful contracting business with Mr. Sherman as its President. In 1896, Mr. McDonough retired when the Company moved its home office to Utica. Mr. Sherman's brothers, the late Hon. James Schoolcraft Sherman, a member of Congress and, later, Vice-President of the United States under the Taft Administration, and Mr. Sanford F. Sherman, succeeded Mr. McDonough as partners in the business.

The Troy Public Works Company constructed a large part of the waterworks for Utica and Norwich, N. Y., as well as public works for Perry and Amsterdam, N. Y., New Britain, Conn., Boston, Pittsfield, and North Adams, Mass., and considerable State work along the Eastern Division of the Barge Canal. The Company also built under contract with the Government the foundation for the United States Gun Factory at Watervliet, N. Y.

Mr. Sherman served as Democratic Mayor of Utica during 1900 and 1901 and again in 1906 and 1907. Under his first régime, $165 000 was appropriated for a new public library and large sums for new school buildings. Provision was made also for new river bridges, for the abolition of railroad grade crossings on the principal streets, for a new union passenger station, a new New York Central Railroad freight house, etc.

On retiring from his second term of office as Mayor of Utica, Mr. Sherman resumed his work with the Troy Public Works Company and completed the plant of the Empire State Power Company, on Schoharie Creek, near Amsterdam, N. Y. From August, 1911, to April, 1915, he held the important position of Chief Engineer of the New York State Conservation Commission.

Mr. Sherman was self-reliant and held strong opinions generally based on firm foundations. Thoroughness was one of his outstanding characteristics and in early manhood he acquired the best and most practical education obtainable. In his railroad building he gained an experience and knowledge of methods that combined to make him a skilled engineer. As a constructor of public works, his thoroughness and his faithful adherence to contracts brought him deserved success. At all times he stood for the best in civic, social, and educational life. As an Engineer, and as Mayor of Syracuse, he gave to his work his best judgment, his most careful consideration, and his honest endeavor.

He was a member of Utica Lodge 47, F. and A. M., the Oneida Historical Society, and the Albany Country Club. He was also a member of the First Presbyterian Church of New Hartford.

On May 20, 1873, he was married to Julia H. Hammond who died on March 13, 1919. He is survived by a daughter, Elizabeth Sherman, a brother, Sanford F. Sherman, and a sister, Mrs. Henry J. Cookinham.

Mr. Sherman was elected a Member of the American Society of Civil Engineers on October 6, 1886.

FRANK SUTTON, M. Am. Soc. C. E.*

DIED JULY 22, 1926.

Frank Sutton, the son of Peter and Jane (Mechling) Sutton, was born in Indiana, Pa., on May 14, 1859. At the age of 20, he entered the Pennsylvania Military College at Chester, Pa., from which he was graduated in 1879 with the degree of Bachelor of Science in Civil Engineering. In 1912, the degree of Master in Civil Engineering was conferred on him by the same institution, and, in 1916, he was elected a Trustee of the College.

Following his graduation, from 1879 to 1886, Mr. Sutton was employed by the Pittsburgh and Western Railroad Company, the old "South Penn" as it was then known, serving as Draftsman, Rodman, Transitman, and Topographer. The "South Penn" Corps set a standard in topographic work that has seldom been equalled, and Mr. Sutton gained topographic experience that afterward brought to him the high position he held in that branch of engineering in the United States Geological Survey.

On April 5, 1886, Mr. Sutton was appointed a Draftsman in the U. S. Geological Survey and, in 1889, he was placed in charge of a topographic field party. His natural ability as a Topographer, combined with his training as an Engineer, attained for him rapid promotions and, in 1906, he reached the grade of Geographer in charge of the Atlantic Division of Topography. It was in this section of the United States, that reaches from Maine to Florida and includes Ohio with the States to the south and west as far as the Mississippi River, that he developed the men under his direction and produced topographic maps of the highest order which are a lasting memorial of his ability.

Frank Sutton was commissioned a Major in the Engineer Officers' Reserve Corps on January 23, 1917, and was called to active duty on June 22, 1917. He was first assigned to special topographic mapping duty with the U. S. Geological Survey, but later was assigned to duty with the 25th, 29th, and 304th Engineers, having had command of the latter. He served in France from August, 1918, to April, 1919, and on the completion of his service with the American Expeditionary Forces, he returned to the United States and was honorably discharged on April 7, 1919. He was then reinstated in his former position of Geographer in the U. S. Geological Survey, which position he held until his retirement, on account of ill health, on March 7, 1924.

Major Sutton's death was probably the result of exposure and sickness at Camp Devens, Massachusetts, during the winter of 1917-18. After a long period of treatment at Johns Hopkins University, where his ailment was diagnosed as amyotrophic lateral sclerosis, he was taken to his home at Indiana, Pa., where for more than two years he received the loving care of his sister and two brothers. His condition gradually grew worse until at the time of his death he was completely paralyzed, although he retained to the end, his mental faculties, his cheerful disposition, and his keen interest in current events.

* Memoir prepared by C. H. Birdseye, M. Am. Soc. C. E.

Major Sutton's stability of character, devotion to duty, shrewd wisdom, and fine sense of humor made for him a host of friends by whom his loyalty and kindly advice will be remembered with deep affection. His death brought to an end a life of constant devotion to his chosen profession of engineering.

Major Sutton never married. A sister, Mrs. W. F. Wettling, and two brothers, Ross M. and Phil M. Sutton, of Indiana, Pa., survive him.

His membership in societies and clubs included the Cosmos Club and Chevy Chase Club, of Washington, D. C., Society of American Military Engineers, Washington Society of Engineers, Geological Society of Washington, American Association for the Advancement of Science, American Forestry Association, and American Federation of Arts.

Major Sutton was elected a Member of the American Society of Civil Engineers on October 3, 1906.

MARK ALBIGENCE WALDO, M. Am. Soc. C. E.*

DIED JULY 16, 1926.

Mark Albigence Waldo, the son of William Williams and Dorothy (Wood) Waldo, was born in Manitowoc, Wis., on August 1, 1859. His early education was obtained in the local public schools, and he was a member of the second class to be graduated from the Manitowoc High School. In 1876, he attended the Academy at Prattsburg, N. Y., and, in 1878, entered the University of Wisconsin and was enrolled in the Engineering Course.

Having completed his Freshman year at the University, he accepted work under the late Grenville Mellen Dodge, Hon. M. Am. Soc. C. E., whose headquarters at that time were at Helena, Mont., on the construction of the Union Pacific Railroad. It was while engaged on this work that Mr. Waldo had his initial experience in bridge building, some of the structures over the Gallatin River and the head-waters of the Missouri River on which he was engaged being particularly difficult from an engineering standpoint. This work resulted in his rapid development as an engineer, with the promise of high rank in the profession which his record amply verified during his years of active practice.

After a year of practical experience, he returned to the University of Wisconsin to complete his engineering course, and was graduated therefrom in 1884 with the degree of Bachelor in Mechanical Engineering.

In September, 1884, Mr. Waldo commenced the practice of engineering by accepting the position of Instrumentman and Pile-Driver Inspector on the construction of the St. Louis River Bridge, at Duluth, Minn., for the Northern Pacific Railroad Company. In 1892, he served as Assistant Engineer on the construction of the Memphis Bridge, at Memphis, Tenn.; and, in 1898, he was Resident Engineer for the Missouri Bridge and Iron Works. In 1899, he was placed in charge of construction by this Company. Mr. Waldo specialized in

* Memoir prepared by W. Lacy Boyd, M. Am. Soc. C. E.

bridge building, particularly in foundations and substructures which gave an outlet for his great ability and resourcefulness, and during his active practice he was associated with engineers who were representative of the best men in the profession, among whom may be mentioned the late George S. Morison and Alfred Noble, Past-Presidents, Am. Soc. C. E.

In 1904, Mr. Waldo retired from the practice of engineering and entered the field of phosphate mining in Polk County, Florida, as General Manager of the Dominion Phosphate Company, with headquarters at Bartow, the center of the pebble phosphate fields of the State. At this time he declined an offer from the Florida East Coast Railroad Company to serve as Chief Engineer on the construction of the "Overseas Railroad" from Miami to Key West.

From the time of his becoming a "Floridian" by adoption and from choice, he made his home in Bartow, taking an active interest in civic affairs, with particular attention to the interests of the Public Library. He was a member of the Library Board from the time of its organization until his death, his work and his advice having been sincerely appreciated by all who came within its influence. It was during his service as a member of the City Council of Bartow that the question of sidewalks came before that body for discussion. Mr. Waldo firmly insisted that a policy should be outlined and a system adopted, based on a comprehensive survey of drainage conditions, grades, and street center lines. This recommendation prevailed, but before volume construction could be started, it was found that the interest created was such that a complete system of street improvement was demanded, and the engineering mind of Mark Albigence Waldo was primarily responsible for blazing the way for lifting the City of Bartow out of the class of sandy little villages into that of a modern Florida city.

He died of apoplexy on July 16, 1926, at his home in Bartow, and interment was made on July 20, at Manitowoc, Wis. The members of his immediate family surviving him are his widow, Mary Agnes (Ott) Waldo, to whom he was married at Jefferson City, Mo., on March 18, 1896; a daughter, Dorothy Wood Waldo, of Bartow; a son, Lieut. George Edward Waldo, U. S. A., of Fort Monroe, Va.; and a granddaughter. He is also survived by two sisters, Mrs. Edward H. Ross of Mansfield, Pa., and Miss Flora Waldo, of Manitowoc, Wis., and a brother, Mr. George E. Waldo, of Chicago, Ill.

Mr. Waldo was a Phi Kappa Psi at the University of Wisconsin; a Mason; a Knight Templar; a member of Egypt Temple, A. A. O. N. M. S., Tampa, Fla.; an active member of the Protestant Episcopal Church; a member of the Yacht and County Club of Tampa, the Golf Club of Bartow, and the Engineers Club of New York, N. Y. He was a man who was quiet and unobtrusive in his views, always courteous, deeply interested in fishing and all outdoor sports, and one who made strong and steadfast friends. The Society has lost a member who was a credit to it; his friends have lost a companion whose place cannot be filled by another.

Mr. Waldo was elected a Member of the American Society of Civil Engineers on May 2, 1900.

FRANK THOMAS WESTCOTT, M. Am. Soc. C. E.*

DIED OCTOBER 22, 1924.

Frank Thomas Westcott was born at Burrillville, R. I., on August 6, 1868. His early education was obtained in the public schools at Milford, Conn., and he received his college training at Brown University, Providence, R. I., from which he was graduated in 1890 with the degree of Bachelor of Philosophy. In 1892, he was graduated from the Massachusetts Institute of Technology, at Cambridge, Mass., with the degree of Bachelor of Science.

During 1892 and 1893, Mr. Westcott held a position with the Triangulation Department of the Board of Survey, Boston, Mass., and in 1893 and 1894 he was Chief Assistant Engineer with C. A. Hicks, Engineer, of Needham, Mass., in which position his principal work consisted in land development.

From 1894 to 1896, he had charge of the outside work for the City Engineer's Office at Central Falls, R. I., which included sewer construction costing about $50 000, and also street construction.

In 1896, Mr. Westcott engaged in private practice as a Civil Engineer with an office in North Attleborough, Mass., which practice he continued until his death on October 22, 1924. From 1899 to 1907 he served as Superintendent of Highways and had charge of all municipal improvements for North Attleborough. In addition, he acted as Engineer for the town in connection with several proposed street railways and also for the Boston and Providence Interurban Railway. At this time he designed and constructed several miles of sewers for surface water and two highway bridges with steel spans and concrete floors.

In 1909, he acquired the business of the R. H. Tingley Company which he conducted in connection with the North Attleborough office. He also made studies for a storage reservoir of between 200 and 300 acres. From 1908 to 1924, Mr. Westcott held the position of Town Treasurer and was also a Member of the Finance Committee for the Town of North Attleborough.

Mr. Westcott was a capable, level-headed engineer, and a good citizen, quiet, efficient and hard-working, to whom many offices of importance were entrusted by his fellow townsmen.

On May 31, 1894, he was married to Elizabeth Nancy Tourtellot who, with a son, survives him.

Mr. Westcott was elected a Member of the American Society of Civil Engineers on January 3, 1911.

CHARLES GRIFFITH YOUNG, M. Am. Soc. C. E.*

DIED JUNE 16, 1926.

Charles Griffith Young, the son of Charles Henry and Marion (Kellogg) Young, was born at Bath, N. Y., on November 1, 1866. He received his education at the Haverling Academy where he took special courses in technical subjects and from which he was graduated in 1885.

* Memoir compiled from information on file at the Headquarters of the Society.

His first work was with the Schuyler Electric and Manufacturing Company, of Hartford and Middletown, Conn., with which Company, in 1886 and 1887, he held the position of Assistant Engineer of installation plants. During this engagement he also took an expert course in every department of the works. From 1888 to 1892, he served as Engineer and General Superintendent for the Mount Morris Electric Light Company, of New York, N. Y. His work at this time consisted of the construction of two central stations and one temporary plant, a complete distribution system underground and overhead.

From 1892 to 1900, except for a short period, Mr. Young acted as Supervising Engineer for the White-Crosby Company and the J. G. White Engineering Company, of New York, N. Y. In this position he made investigations and reports on electric lighting systems, power plants, steam railways, and electric railways in Baltimore, Md., Richmond, Va., Washington, D. C., Toledo, Ohio, Detroit, Mich., and in Haiti. He was also engaged on the construction of some of this work.

For a short time in 1898 and 1899 Mr. Young served as Supervising Engineer for the New York Heat, Light, and Power Company, his work including plant extension and operation. He also investigated and reported on the electric lighting situation in New York and on proposed steam railway, electric railway, and electric lighting installations in Porto Rico and in Havana, Cuba.

From 1900 to 1902 Mr. Young held the position of Engineer for J. G. White, Ltd., of London, England, and as such was engaged in investigations and reconnaissance work, design, and specifications for electric railways and lighting and power systems. During this period he estimated the cost and earnings of a complete electric railway equipment for Auckland, New Zealand. He also made estimates and investigated costs and earnings and made general plans for electric railways, lighting and power plants at Para, Brazil; Montevideo, Uruguay; Buenos Aires and Rosario, Argentine Republic; and in the Philippine Islands.

He served as General Construction Manager from 1902 to 1909 for J. G. White and Company, in New York, on plant specifications, layout designs, and the construction of electric railways, power plants, and steam railways. He was also engaged on harbor and sea-wall contracts, dredging, water-works, estimates, analyses, and recommendations on the operation and equipment of properties. From 1892 to 1909, more than 2 000 miles of electric railways were built under his direction in the United States and foreign countries.

From 1909 to December, 1917, Mr. Young carried on a general independent practice as a Consulting Engineer, with an office in New York. He specialized in investigations and reports of professional undertakings, both proposed and actual; as an expert before judiciaries and commissions on valuations, etc.; and on operating conditions of steam and electric railroads, water and rail terminals, hydro-electric power, etc.

In 1918, he served as Engineer for Ford, Bacon, and Davis, Incorporated, of New York, on contract negotiations, and, from 1919 to 1921, as Manager for John N. Willys Export Corporation, of New York, in which capacity he was engaged chiefly on export engineering problems.

In 1922, Mr. Young returned to private practice and organized the C. G. Young Company, Incorporated, of which he was President, which position he held at the time of his death. His engineering activities extended over a wide range, including Canada, South America, Siberia, and various European countries, in the course of which he had traveled three times around the world. He died suddenly in Porto Rico on June 16, 1926, while on a professional engagement.

On October 21, 1890, Mr. Young was married to Bessie Curtis Leonard, of New York, N. Y., who, with a daughter, Elizabeth D. Young, survives him.

He was the author of a valuable paper entitled "Logical Basis for Valuations", which was printed in pamphlet form and which also appeared in *Electric Traction* of January 21, 1911.

As an engineer Mr. Young was always courageous and forward-looking. In addition to practical engineering skill, he was distinguished by the happy faculty of always seeing beyond obstacles, and of viewing optimistically any work before him, with an inspired faith in its successful completion.

Those close to him knew him as an unselfish, devoted friend, whose loyalty ripened with the years. Even his casual acquaintances knew him as a man of high principle and fine character, and they remember him now for those lovable qualities that kept expressing themselves through his contagious smile.

He was a member of the American Institute of Electrical Engineers; National Electric Light Association; New York Electrical Society; American Electric Railway Transportation and Traffic Association; American Electric Railway Engineering Association; Pan American Society; Pan American Chamber of Commerce; Steuben Society; Engineers' Club of New York; New York Railroad Club; Indian House Club; and the Circumnavigators.

Mr. Young was elected an Affiliate of the American Society of Civil Engineers on March 6, 1894, and a Member on March 8, 1921.

WILLIAM HANNUM, Assoc. M. Am. Soc. C. E.*

DIED MAY 8, 1925.

William Hannum, the son of Preston E. and Charlotte E. (Philips) Hannum, was born on March 26, 1890, at Christiana, Pa. He received his early education in the schools of that town, and was graduated from the High School in 1906.

During the summer months of 1906, Mr. Hannum made his first acquaintance with engineering work as Rodman with an engineering corps on the construction of an earth embankment reservoir at Blairsville Intersection, Pa. From the very first his ability to handle men was evident, for at odd times he acted as Foreman over a gang of laborers and before long became the regular Labor Foreman.

In September of that year, however, Mr. Hannum decided that an engineering education would be an advantage if he expected to advance in this par-

* Memoir prepared by H. G. MacMurchy, Constr. Supt., Phœnix Utility Co., Memphis, Tenn.

ticular line of endeavor; so he left his position for two years of preparatory work at the West Chester State Normal School. The vacation months of 1907 found him back with the same Superintendent, as Labor Foreman and Team Boss, on a masonry reservoir at Johnstown, Pa., for the American Pipe Manufacturing Company, of Philadelphia, Pa.

The following four years were spent at Pennsylvania State College, from which Mr. Hannum was graduated with honors in 1912. As with most students, he spent his vacations at whatever work came to hand. One summer he was employed on a cattle steamer to England, where he spent two months in sight-seeing; another vacation was spent as a laborer in a lumber mill in Pennsylvania; and still another in the grain fields of the Middle West, and also as a clerk in a hotel at a fashionable summer resort in Pennsylvania.

After graduating in 1912, Mr. Hannum obtained a position as Engineer with the J. G. White Engineering Corporation, and was sent to Parkesville, Tenn., on a hydro-electric development. Although rated as an Engineer, he was made Material Tracer, and for eight months traveled through the Southern States, tracing shipments of construction material. This work, however, was shut down before completion, and Mr. Hannum was forced to seek other employment.

In August, 1913, he obtained a position as Labor Foreman with Latta and Terry, Contractors, of Philadelphia, on the foundations of the elevated railway escalator at Delaware and Market Streets, Philadelphia. Later, he was in charge of the construction of a sand and gravel washing plant at Bridgeboro, N. J., for the same firm.

The stringent times of January, 1914, found Mr. Hannum out of work. After a short time he accepted a position as Bond Salesman for N. W. Halsey, of Philadelphia. In this line of endeavor he proved to be very successful, but since engineering work had the greatest appeal, he resigned on September 1, 1916, to form a partnership under the name of McCool and Hannum, Contractors and Engineers. This partnership consisted of Messrs. Paul H. Turnbull, John McCool, and William Hannum, with offices in Philadelphia. The engineering operations were confined to that vicinity and consisted of building sewers for the City of Philadelphia; gas lines for the Suburban Gas and Electric Company; and several buildings of moderate size. The entrance of the United States into the World War, however, with the attendant rise in costs of labor and material, found the new firm unable to weather the financial storm and, in June, 1917, the partnership was dissolved.

On October 1, 1917, Mr. Hannum returned to the J. G. White Engineering Corporation, and was sent to U. S. Nitrate Plant No. 1, this work having just been started at Sheffield, Ala. The first problem was to organize a construction crew and Mr. Hannum, as Assistant Superintendent, was detailed to this task. He also acted as Special Purchasing Agent of Construction Plant and Equipment, later organizing the Purchasing Department. In May, 1918, he became Division Superintendent of Construction, taking charge of the construction of houses, school buildings, streets, sidewalks, sewers, sewage disposal plant, water supply system, water filtration plant, and other activities in

the permanent village for the operating employees of the plant. This work was completed in the summer of 1919, but Mr. Hannum remained in charge of the force that closed down operations, returning to the New York Office in November, 1919.

His next assignments for the J. G. White Engineering Corporation, consisted of a one-story brick and steel warehouse at Pittsburgh, Pa., for the General Chemical Company; a wireless telegraph receiving station at Lakewood, N. J., and, later, extensions to a pulp mill at Chandler, Que., Canada. When this last work was completed in June, 1921, the ensuing business depression found Mr. Hannum out of a position. He, however, became connected temporarily with Rogers and Hagerty, Incorporated, of New York, N. Y., which firm was working on the Hill-to-Hill Bridge at Bethlehem, Pa. As this work was not in line with his ability, he resigned in April, 1922, to take a position with the Utilities North American Corporation, of Buffalo, N. Y., to sell stock. This proved to be a bad venture and in February, 1923, Mr. Hannum returned to his old connection with the J. G. White Engineering Corporation, and was sent to Ronceverte, W. Va., on the construction of a steam power station and the installation of equipment for the Virginia Western Power Company. The next assignment was the construction of the village for the operating employees of the East Pennsylvania Electric Company at Pine Grove, Pa. This work was completed in February, 1924.

In April, 1924, Mr. Hannum resigned his position with the J. G. White Engineering Corporation to accept employment as Construction Superintendent with the Phœnix Utility Company, and was sent to Memphis, Tenn. The work was the completion of a steam electric station for the Memphis Power and Light Company, and consisted of the installation of a 20 000-kw. turbo-generator, with its auxiliaries; also the moving of two smaller units from another station to be used for stand-by service; the building of two substations in Memphis; and also similar plant construction work in Jackson, Tenn., and Little Rock, Ark.

It was while in the midst of this program that Mr. Hannum was called to an untimely death. The Engineers Club of Memphis called a Convention of the Engineers of the Mid-South for May 6 to 8, 1925, and Mr. Hannum, as a member of the Sight-Seeing Trip Committee, assisted in entertaining the visitors, opening the Power Plant and its construction work for their inspection. After the tour of the industries of the city, those attending the Convention were taken aboard two Government boats, the *Choctaw* and *Norman*, and carried down the Mississippi River about 25 miles to inspect some revetment work that was being done by the Government Engineers. The trip down the river to Cow Island Bend was delightful and the inspection of the willow mats very interesting.

Before starting on the return trip, word was passed around that all civil engineers would transfer to the *Norman*, to discuss the formation in Memphis of a Local Section of the American Society of Civil Engineers. Mr. Hannum had already returned to the *Choctaw* with several friends, and was comfortably seated on the upper deck. A fellow engineer called for him and Mr. Hannum

transferred to the *Norman*, so as to be present at the conference and join in the discussion.

The *Choctaw* took the lead on the return trip and, being a faster boat, in a little more than an hour had passed out of sight, hidden by the bends in the river. By the time the conference was in full swing on the upper deck of the *Norman*, it was noticed that the boat was listing slightly and a few minutes later the pilot sent orders for the passengers to move to the side of the boat. Shortly after, a second request was made for the passengers to shift positions, only to be followed in a few minutes by a decided list to starboard, which continued slowly until the boat had turned completely over. Mr. Hannum, with many others, climbed aboard the up-turned boat and rendered such assistance as he could to those in a more perilous position. It was at this time that he performed a noble deed that is best described by Alfred M. Lund, M. Am. Soc. C. E., of Little Rock, Ark., an eye-witness:

"When the wooden superstructure of the *Norman* came up and put the boat on its side it brought to the surface the body of a woman. Mr. Hannum immediately jumped from the hull of the boat down into the water, which was still over the side of the boat, and calling for help endeavored to drag the woman's body to a position which at that time appeared to be safer. I was about 10 ft. down stream and seeing he was unable to carry her by himself I went to his assistance. He had secured the body by the shoulders and was half lifting and half dragging it to the exposed part of the hull when suddenly the boat went down and that was the last I saw of them."

In describing the incident the *Commercial Appeal* fitly claimed that, "he died a hero". Eighteen days later, Mr. Hannum's body was found fifteen miles below the scene of the disaster. The body was taken to Florence, Ala., for interment.

A disaster that removed twelve engineers, five women of their party, a boy, and five members of the crew, was a sad blow to the Engineering Fraternity of the Mid-South.

Beloved by his associates and friends, Mr. Hannum will be sadly missed and his death leaves a gap that will be hard to fill. He was a Tau Beta Pi of Beta Chapter of Pennsylvania State College; a member of the Engineers' Club of Memphis, and a member of Zamora Temple, A. A. O. N. M. S., of Birmingham, Ala.

Mr. Hannum was married on September 14, 1919, to Annie Lipscomb, of Florence, Ala., who, with a daughter, Charlotte, three years old, survives him.

Mr. Hannum was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919.

JOHN McKNIGHT SOUTHGATE, Assoc. M. Am. Soc. C. E.*

DIED JULY 6, 1926.

John McKnight Southgate was born at Rolla, Mo., on September 17, 1881, the third child and only son of Judge W. Wallace and Annie McKnight

* Memoir prepared by Ray Dennison, Esq., St. Louis, Mo., and R. W. Abbott, Esq., Pontiac, Mich.

(Black) Southgate. On his father's side, he was a descendant of a long line of New England ancestors. His mother's parents were among the early English settlers of Missouri, her grandparents having immigrated from Augusta County, Virginia, in 1815, taking three months to make the trip by wagon and flat-boat down the Ohio River.

Mr. Southgate's earlier education was obtained in the Rolla Public Schools with the exception of one year (1896-97) which he spent at St. Mark's School at Portland, Mo. His education was completed at the Missouri School of Mines and Metallurgy at Rolla, where he was a football star and a member of the Sigma Nu Fraternity.

After leaving college Mr. Southgate engaged in railroad engineering. From 1902 to 1908 he was employed by the St. Louis and San Francisco and the Missouri Pacific Railroad Companies, respectively, as Resident Engineer on construction and as Assistant Engineer on location and maintenance. From 1908 to 1911, he was connected with the Sewer Department of St. Louis, Mo., as Engineer of Construction of the River des Peres and other Sewers.

In 1911, he organized a construction company of which he was President and engaged in paving and sewer contracting work until 1913, when he returned to private practice as a Consulting Engineer with offices at Rolla. From 1913 to 1923, in conjunction with his consulting practice, Mr. Southgate held the offices of Surveyor and Highway Engineer of Phelps County, Missouri, City Engineer of Rolla, and Resident Engineer of the Missouri State Highway Commission. His consulting practice consisted mainly of municipal and highway work.

In 1923, he returned to the Department of Sewers of the City of St. Louis, in charge of surveys for the $11 000 000 River des Peres Sewer System and remained in that position for one year. In 1924, he was placed in charge of pier construction and harbor improvements in Mobile Bay, for the Mobile and Ohio Railroad Company. He finished this work in January, 1925, and from that time until February, 1926, acted as Division Engineer of Construction and Maintenance of Way, for the same Company.

In February, 1926, Mr. Southgate became connected with the Ozark Pipe Line Corporation (Roxana Petroleum Corporation) as Chief Engineer of Construction. While engaged with his duties in this work he was stricken with acute appendicitis on July 2, 1926, and died four days later in a hospital in Hammond, Ind. His body was taken to his home in Rolla, for interment. He is survived by his widow, Elizabeth Baker Southgate, and one daughter, Mrs. Robert W. Abbett.

Mr. Southgate's entire life was spent in the active practice of engineering. He was endowed with those inborn qualities of character so essential to the engineer—sterling integrity, cool and impartial judgment, resourcefulness, and a rare faculty for inspiring confidence and loyalty in his peers and subordinates.

On account of his preference for active field practice he did not always cast his lines in pleasant places, but he bore the privations and discomforts incidental to the accomplishment of his work with unfailing cheerfulness and

saw in obstacles only the satisfaction to be found in surmounting them. Capable of infinite patience in attention to the details of a problem, he was quick to reprimand carelessness or laxity in the line of duty, but he was no less ready to recognize and commend worthy effort on the part of others.

The charm of Mr. Southgate's personality made him popular with all classes of people. He was especially loved and respected by those younger men of the profession who were fortunate enough to be numbered among his friends. To several of these he gave the opportunity and assistance which launched them into the practice of engineering.

The place he held in the hearts of his friends can never be filled, but his memory remains as an ideal of the God-fearing gentleman and the engineer.

Mr. Southgate was a member of the American Association for the Advancement of Science and the Missouri Highway Association, having served as President of latter organization in 1919 and 1920. He was also a member of the Executive Committee of the Missouri Good Roads Federation, the organization that promoted and passed the $60 000 000 road bond issue for that State.

He was a member of the Masonic Fraternity and had served as Master of Rolla Lodge No. 213, F. and A. M.; High Priest of Rolla Chapter of Royal Arch Masons; Commander of Rolla Commandery, Knights Templars; Worthy Patron of Rolla Chapter, Order of the Eastern Star; and had also held office in Rolla Council of Royal and Select Masters and Moolah Temple of the Mystic Shrine. Throughout his life he was a devout member of the Protestant Episcopal Church.

Mr. Southgate was elected an Associate Member of the American Society of Civil Engineers on July 11, 1921.